Appendix E

Report of Geotechnical Investigation
Proposed Todd Cancer Institute

REPORT OF GEOTECHNICAL INVESTIGATION
PROPOSED TODD CANCER INSTITUTE

LONG BEACH MEMORIAL MEDICAL CENTER
2801 ATLANTIC AVENUE
LONG BEACH, CALIFORNIA

Prepared for:

LONG BEACH MEMORIAL MEDICAL CENTER

Long Beach, California

Prepared by:

MACTEC Engineering and Consulting

October 27, 2004

Project 4953-04-2891

MACTEC

MACTEC

October 27, 2004

Mr. Pat Johner, Vice President
Long Beach Memorial Medical Center
2801 Atlantic Avenue
Post Office Box 1428
Long Beach, California 90801-1428

Subject: **Report of Geotechnical Investigation**
Proposed Todd Cancer Institute Building
Long Beach Memorial Medical Center
2801 Atlantic Avenue
Long Beach, California
MACTEC Project 4953-04-2891

Dear Mr. Johner:

We are pleased to submit the results of our geotechnical investigation for the proposed Todd Cancer Institute building to be constructed at the Long Beach Memorial Medical Center in Long Beach, California. This investigation was conducted in general accordance with our proposal dated September 16, 2004 as authorized by you on the same date.

The scope of our services was planned with Mr. John King of Adams Project Management Consulting. Mr. Marc Davidson of Cannon Design furnished us with a site plan for the project. Mr. Joseph Stewart of KPFF Consulting Engineers advised us of the structural features of the proposed Todd Cancer Institute.

The results of our investigation and design recommendations are presented in this report. Please note that you or your representative should submit copies of this report to the appropriate governmental agencies for their review and approval prior to obtaining a building permit.

MACTEC Engineering and Consulting, Inc.
200 Citadel Drive • Los Angeles, CA 90040
323-889-5300 • Fax: 323-721-6700

It has been a pleasure to be of professional service to you. Please call if you have any questions or if we can be of further assistance.

Sincerely,

MACTEC Engineering and Consulting, Inc.

J. Adolfo Acosta, Ph.D.
Senior Engineer

Susan F. Kirkgard
Senior Engineering Geologist

Jake Kharraz
Principal Engineer

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(2 copies submitted)

Attachments

cc: (4) Adams Project Management Consulting, LLC
Attn: Mr. John R. King

(2) Cannon Design
Attn: Mr. Marc Davidson

(1) KPFF Consulting Engineers
Attn: Mr. Joseph Stewart

REPORT OF GEOTECHNICAL INVESTIGATION
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LONG BEACH MEMORIAL MEDICAL CENTER
2801 ATLANTIC AVENUE
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Los Angeles, California

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TABLE OF CONTENTS

Page

LIST OF TABLES AND FIGURES iii
SUMMARY iv
1.0 SCOPE 1
2.0 PROJECT DESCRIPTION 2
3.0 SITE CONDITIONS 3
4.0 EXPLORATIONS AND LABORATORY TESTS 3
5.0 SOIL CONDITIONS 4
6.0 GEOLOGY 5
 6.1 GEOLOGIC SETTING 5
 6.2 GEOLOGIC MATERIALS 5
 6.3 GROUND WATER 6
 6.4 FAULTS 6
 6.5 GEOLOGIC-SEISMIC HAZARDS 11
 6.6 CONCLUSIONS 15
7.0 GROUND MOTION STUDY 16
8.0 RECOMMENDATIONS 17
 8.1 FOUNDATIONS 17
 8.2 SITE COEFFICIENT AND SEISMIC ZONATION 19
 8.3 FLOOR SLAB SUPPORT 20
 8.4 PAVING 21
 8.5 GRADING 23
 8.6 GEOTECHNICAL OBSERVATION 25
9.0 BASIS FOR RECOMMENDATIONS 26
10.0 BIBLIOGRAPHY 27

APPENDIX: EXPLORATIONS AND LABORATORY TESTS

LIST OF TABLES AND FIGURES

Table

1	Major Named Faults Considered to be Active in Southern California
2	Major Named Faults Considered to be Potentially Active in Southern California
3	List of Historic Earthquakes
4	Pseudospectral Velocity in Inches/Second
5	Pseudospectral Acceleration in g

Figure

1	Vicinity Map
2	Plot Plan
3	Geologic Section
4	Local Geology
5	Regional Faults
6	Regional Seismicity
7	Seismic Hazards Zone Map
8.1	Response Spectra – Pseudospectral Acceleration, Design Basis Earthquake
8.2	Response Spectra – Pseudospectral Acceleration, Upper Bound Earthquake

SUMMARY

We have completed our geotechnical investigation of the site of the proposed Todd Cancer Institute building to be constructed at the Long Beach Memorial Medical Center. Our subsurface explorations, engineering analyses, and foundation design recommendations are summarized below.

We explored the soil conditions by drilling five borings at the site; fill soils, up to 3 feet thick, were encountered locally in our borings. The natural soils consist of Pleistocene age alluvial deposits that consist of silty sand and sandy silt with some clay. The natural soils are firm and dense.

The existing fill soils are not considered suitable for support of the proposed building or the building floor slab. The natural soils at the site are generally firm and dense. The building can be supported on spread footings established in the firm and dense undisturbed natural soils. If the recommendations on grading are followed, the floor slab can be supported on grade. As an alternative, a mat-type foundation could be used as a foundation support.

Based on the available geologic data, active or potentially active faults with the potential for surface fault rupture are not known to be located beneath or projecting toward the site. In our opinion, the potential for surface rupture at the site due to fault plane displacement propagating to the ground surface during the design life of the project is considered low. Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in Southern California and the effects of ground shaking can be mitigated by proper engineering design and construction in conformance with current building codes and engineering practices.

The site is not located within an area identified as having a potential for liquefaction and the depth to ground water beneath the site is greater than 15 meters (50 feet). Therefore, the potential for liquefaction at the site is low. The site is relatively level and the absence of nearby slopes precludes slope stability hazards. The potential for other geologic hazards such as tsunamis, inundation, seiches, flooding, and subsidence affecting the site is considered low.

The site is located within the Long Beach Oil Field and there is a potential for methane and other volatile gases to occur at the site. Also, abandoned oil wells are documented within 0.3 kilometer (950 feet) of the site. Therefore, there is a potential that abandoned oil wells could be encountered during the development of the site.

1.0 SCOPE

This report provides the results of a geotechnical investigation performed for the proposed Todd Cancer Institute Building. The location of the site is depicted in Figure 1, Vicinity Map. The locations of the proposed building and our exploration borings are depicted in Figure 2, Plot Plan.

This investigation was authorized to determine the static physical characteristics of the soils at selected locations, and to provide recommendations for foundation design and floor slab support for the proposed new development. In addition, we were to update our previous geologic-seismic hazards evaluation and the ground motion study for the site. More specifically, the scope of the investigation included the following:

- Subsurface explorations to determine the nature and stratigraphy of the subsurface soils, and to obtain undisturbed and bulk samples for laboratory observation and testing.
- A geologic-seismic hazards evaluation in conformance with Tithe 24 of the California Code of regulations and with the California Geological Survey checklist for review of geologic-seismic reports for California Public Schools, Hospitals, and Essential Services Buildings (CGS Note 48).
- Evaluation of the liquefaction potential of the soils underlying the site.
- Ground motion study.
- Laboratory testing of soil samples for determination of the static physical soil properties.
- Corrosion studies to determine the presence of potentially corrosive soils.
- Engineering evaluation of the geotechnical data to develop recommendations for design of foundations and walls below grade, for floor slab support, and for earthwork for the proposed building.
- Preparation of a formal report summarizing the data collected and presenting our design recommendations.
- Subgrade preparation, floor slab support, and paving recommendations.
- Design of minor structures.

- Grading, including site preparation, the placing of compacted fill, and quality control measures relating to earthwork.

The assessment of general site environmental conditions for the presence of pollutants in the soils and ground water at the site was beyond the scope of this investigation.

Our recommendations are based on the results of our current exploration borings, laboratory tests, and appropriate engineering analyses. The results of the current field explorations and laboratory tests are presented in the Appendix. The results of the soil corrosivity study are presented at the end of the Appendix.

The information in this report represents professional opinions that have been developed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for the Long Beach Memorial Medical Center and their design consultants to be used solely in the design of the proposed Todd Cancer Institute Building. The report has not been prepared for use by other parties, and may not contain sufficient information for purposes of other parties or other uses.

2.0 PROJECT DESCRIPTION

The proposed Todd Cancer Institute building is prepared to be constructed at the southeast corner of Spring Street and Long Beach Boulevard and within the northwest portion of the Long Beach Memorial Medical Center campus. The location of the proposed building relative to existing structures and streets is shown in plan in Figure 2. Based on the information provided by the structural engineer, the proposed building will be three stories high with no basement planned. A linear accelerator unit (LAU) will be placed within the ground floor level of the building. The building will be of steel frame construction and a metal deck roof covered with concrete. In addition, the linear accelerator unit will have a 2-foot thick reinforced concrete walls, slab and roof. We have been informed by the project structural engineer that a seismic joint will be placed between the LAU and the remainder of the building.

The building will be an out-patient building and the project plans and reports will be reviewed by the City of Long Beach.

The anticipated structural loading for the building and LAU, provided by the structural engineer, is as follows:

Building (dead plus live load, approximately 1/3 is live load):
Interior Column Loads: 425 to 450 kips
Exterior Column Loads: 250 to 275 kips
Linear Accelerator Unit (dead plus live load, approximately 1/3 is live load):
Unit Load per unit length: 8 to 10 kips / lineal foot of wall
Overall soil pressure of Building (dead plus live load, approximately 1/3 is live load):
500 pounds per square foot
Overall soil pressure of Linear Accelerator Unit:
Dead Load: 500 to 600 pounds per square foot
Live load: 50 pounds per square foot.

3.0 SITE CONDITIONS

The subject site is located at the southeast corner of Spring Street and Long Beach Boulevard and is part of a large paved parking lot. The site is relatively flat with only a 1- to 2-foot difference in elevation across the site.

4.0 EXPLORATIONS AND LABORATORY TESTS

The soil conditions beneath the site were explored by drilling 5 borings to depths of about 35 to 62 feet below the existing grade at the locations illustrated in Figure 2. Details of the explorations and the logs of the borings are presented in the Appendix.

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to determine the pertinent engineering properties of the foundation soils. The following tests were performed:

- Moisture content and dry density determinations
- Direct shear
- Consolidation
- Stabilometer (R-Value)
- Compaction
- Expansion Index

All testing was done in general accordance with applicable ASTM specifications. Details of the laboratory testing program and test results are presented in the Appendix. Corrosion studies were performed by M. J. Schiff & Associates and an R-value test was performed by LaBelle Marvin; the results are also presented in the Appendix.

5.0 SOIL CONDITIONS

Fill soils up to 3 feet thick, were encountered in the borings. The fill soils consist of silty sand and sandy silt with some clay and are not uniformly well compacted. Deeper fill could occur between borings.

The natural soils consist of alluvial deposits that include predominantly medium dense to very dense silty sands and stiff to hard sandy silts and some interbedded stiff to very stiff clayey layers.

Water was encountered in Boring 3 and Boring 2 at depths of 16.2 and 16.7 meters (53 and 55 feet) beneath the existing ground surface, respectively. The depth to ground water encountered in our borings is consistent with the reported regional ground-water level that is at a depth greater than 15 meters (50 feet).

The corrosion studies indicate that the on-site soils are corrosive to ferrous metals and aggressive to copper and would have negligible sulfate attack on portland cement concrete. The report of corrosion studies presented in the Appendix should be referred to for a discussion of the corrosion potential of the soils, and for potential mitigation measures.

6.0 GEOLOGY

6.1 GEOLOGIC SETTING

The site is located on the Long Beach Plain adjacent to the Newport-Inglewood uplift, a northwest-trending structural zone expressed at the surface by a series of discontinuous low hills including Signal Hill located southeast of the site.

Regionally, the site is located within the Peninsular Ranges geomorphic province. This province is bounded by the Santa Monica, Hollywood, Raymond, Sierra Madre, and Cucamonga fault zones on the north, the San Andreas fault zone on the east, the Pacific Ocean coastline on the west, and extends to the Mexican border and beyond, on the south. The province is characterized by elongate northwest-trending mountain ridges separated by straight-sided sediment-filled valleys. The northwest trend is further reflected in the direction of the dominant geologic structural features of the province that are northwest to west-northwest trending folds and faults, such as the nearby Cherry Hill fault of the Newport-Inglewood fault zone, located approximately 0.2 kilometer northeast of the site.

The inferred subsurface distribution of geologic materials that were encountered in our borings at the site is illustrated in Figure 3, Geologic Section. The relationship of the site to local geologic features is depicted in Figure 4, Local Geology, and the faults in the vicinity of the site are shown in Figure 5, Regional Faults. Figure 6, Regional Seismicity, shows the locations of major faults and earthquake epicenters in Southern California. Figure 7, Seismic Hazards Zone Map, shows the location of the site relative to State-designated seismic hazards zones.

6.2 GEOLOGIC MATERIALS

The site is underlain by predominantly Pleistocene age alluvial deposits (California Division of Mines and Geology, 1998). Based on the materials encountered in our borings, artificial fill locally mantles the alluvial deposits consisting of silty sand and sandy silt with some clay. The fill was encountered to a maximum depth of 0.9 meter (3 feet) beneath the existing ground surface. However, deeper fill could be present at the site between boring locations. The Pleistocene age alluvial deposits were encountered to the maximum depth of our borings (62 feet) and consist of poorly bedded silty sand, sandy silt and sand with some layers of clayey silt and silty clay.

6.3 GROUND WATER

The site is located in Section 24 of Township 4 South, Range 13 West, within the West Coast Hydrologic Subarea in the Los Angeles-San Gabriel River Hydrologic Unit. Ground-water level contour maps prepared by Bureau of Engineering – City of Long Beach (1988) and the County of Los Angeles (1990) indicate that the depth to ground water in the site vicinity is greater than 15 meters (50 feet) beneath the existing ground surface. A historic high ground-water contour map prepared by the California Geological Survey (formerly the California Division of Mines and Geology, 1998) for the Long Beach area does not depict contours in the area of the site due to the lack of available data.

Water was encountered in Boring 3 and Boring 2 at depths of 16.2 and 16.7 meters (53 and 55 feet) beneath the existing ground surface, respectively. The depth to ground water encountered in our borings is consistent with the reported regional ground-water level that is at a depth greater than 15 meters (50 feet).

6.4 FAULTS

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly the California Division of Mines and Geology) for the Alquist-Priolo Earthquake Fault Zoning Program (Hart, 1999). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault is a fault that has demonstrated surface displacement of Quaternary age deposits (last 1.6 million years). Inactive faults have not moved in the last 1.6 million years. A list of nearby active faults and the distance in kilometers between the site and the nearest point on the fault, the maximum magnitude, and the slip rate for the fault is given in Table 1. A similar list for potentially active faults is presented in Table 2. The faults in the vicinity of the site are shown in Figure 5.

Active Faults

Newport-Inglewood Fault Zone

The closest active fault to the site is the Cherry Hill fault of the Newport-Inglewood fault zone located approximately 0.2 kilometer to the northeast. The Cherry Hill fault trends northwest from the west flank of Signal Hill to the Los Angeles River where its surface trace becomes less distinct. This fault is one of a series of discontinuous northwest-trending en echelon (laterally stepping) faults extending from Newport Beach to Beverly Hills that comprise the Newport-Inglewood fault zone. The Newport-Inglewood fault zone is one of several large predominately right-lateral strike-slip fault zones that parallel the San Andreas fault in southern California. Deformation within the Newport-Inglewood fault zone is expressed at the surface by a line of geomorphically young, anticlinal hills and mesas, including Signal Hill located east-southeast of the site. The Newport-Inglewood fault zone has been a zone of tectonic deformation since Miocene time with recurrent movement during late Tertiary and Quaternary time (Wissler, 1943). Fault-plane solutions for 39 small earthquakes (between 1977 and 1985) show mostly strike-slip faulting with some reverse faulting along the north segment (north of Dominguez Hills) and some normal faulting along the south segment (south of Dominguez Hills to Newport Beach) (Hauksson, 1987). The 1920 Inglewood and 1933 Long Beach earthquakes (magnitudes 4.9 and 6.3, respectively) resulted from movement on the Newport-Inglewood fault zone. Investigations by Law/Crandall (1993) in the Huntington Beach area indicate that the North Branch segment of the Newport-Inglewood fault zone offsets Holocene age alluvial deposits in the vicinity of the Santa Ana River.

Palos Verdes Fault Zone

Studies by Stephenson et al. (1995) indicate that there are several active on-shore splays of the Palos Verdes fault zone. Based on his study, which included geophysical surveys, aerial photograph interpretation, and limited fault trenching, the nearest splay of the active Palos Verdes fault zone is located about 10½ kilometers southwest of the site. The geophysical data indicates that the dip of the fault ranges from near vertical to 55 degrees to the southwest (Stephenson et al., 1995). Vertical separations up to about 1,800 meters occur across the fault at depth. However, strike-slip movement is indicated by the configuration of the basement surface and lithologic changes in the Tertiary age rocks across the fault. The geophysical data also indicates an offset at the base of the offshore Holocene age deposits (Clarke et al., 1985). However, no historic large magnitude earthquakes are associated with this fault.

San Andreas Fault Zone

The active San Andreas fault zone is located about 78 kilometers northeast of the site. This fault zone, California's most prominent geological feature, trends generally northwest for almost the entire length of the state. The southern segment, which includes the Mojave segment, is approximately 450 kilometers long and extends from the Transverse Ranges west of Tejon Pass on the north to the Mexican border and beyond on the south. Wallace (1968) estimated the recurrence interval for a magnitude 8.0 earthquake along the entire fault zone to be between 50 and 200 years. Sieh (1984) estimated a recurrence interval of 140 to 200 years. The 1857 Fort Tejon earthquake was the last major earthquake along the San Andreas fault zone in Southern California.

Blind Thrust Fault Zones

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3 kilometers. These faults do not present a potential surface fault rupture hazard. However, the following described blind thrust faults are considered active and potential sources for future earthquakes.

Puente Hills Blind Thrust

The Puente Hills Blind Thrust (PHBT) is defined based on seismic reflection profiles, petroleum well data, and precisely located seismicity (Shaw and others, 2002). This blind thrust fault system extends eastward from downtown Los Angeles to Brea (in northern Orange County). The PHBT includes three north-dipping segments, named from east to west as the Coyote Hills segment, the Santa Fe Springs segment, and the Los Angeles segment. These segments are overlain by folds expressed at the surface as the Coyote Hills, Santa Fe Springs Anticline, and the Montebello Hills. The Santa Fe Springs segment of the PHBT is believed to be the causative fault of the October 1, 1987 Whittier Narrows Earthquake (Shaw and others, 2002). The vertical surface projection of

PHBT is located approximately 11½ kilometers northeast of the site at its closest point. Postulated earthquake scenarios for the PHBT include single segment fault ruptures capable of producing an earthquake of magnitude 6.6 (Mw) and a multiple segment fault rupture capable of producing an earthquake of magnitude 7.1 (Mw). The PHBT is not exposed at the ground surface and does not present a potential for surface fault rupture. However, based on deformation of late Quaternary age sediments above this fault system and the occurrence of the Whittier Narrows earthquake, the PHBT is considered an active fault capable of generating future earthquakes beneath the Los Angeles Basin. An average slip rate of 0.7 mm/yr and a maximum magnitude of 7.1 are estimated by the California Geological Survey (2003) for the Puente Hills Blind Thrust.

Upper Elysian Park Fault

The Upper Elysian Park fault is a blind thrust fault that overlies the Los Angeles and Santa Fe Springs segments of the Puente Hills Blind Thrust (Oskin et al., 2000 and Shaw et al., 2002). The eastern edge of the Upper Elysian Park fault is defined by the northwest-trending Whittier fault zone. The vertical surface projection of the Upper Elysian Park fault is approximately 25 kilometers north of the site at its closest point. Like other blind thrust faults in the Los Angeles area, the Upper Elysian Park fault is not exposed at the surface and does not present a potential surface rupture hazard; however, the Upper Elysian Park fault should be considered an active feature capable of generating future earthquakes. An average slip rate of 1.3 mm/yr and a maximum magnitude of 6.4 are estimated by the California Geological Survey (2003) for the Upper Elysian Park fault.

San Joaquin Hills Thrust

Until recently, the southern Los Angeles Basin has been estimated to have a low seismic hazard relative to the greater Los Angeles region (Working Group on California Earthquake Probabilities, 1995; Dolan et al., 1995). This estimation is generally based on the fewer number of known active faults and the lower rates of historic seismicity for this area. However, several recent studies by Grant et al. (2000, 2002) suggest that an active blind thrust fault system underlies the San Joaquin Hills. This postulated blind thrust fault is believed to be a faulted anticlinal fold, parallel to the Newport-Inglewood fault zone (NIFZ) but considered a distinctly separate seismic source (Grant et al., 2002). The recency of movement and Holocene slip rate of this fault are not known. However, the fault, if it exists, has been estimated to be capable of a Magnitude 6.8 to 7.3 earthquake (Grant et al., 2002). This estimation is based primarily on coastal geomorphology and age-dating of marsh deposits that are elevated above the current coastline.

The vertical surface projection of the San Joaquin Hills Thrust is approximately 27 kilometers southwest of the site at the closest point. This thrust fault is not exposed at the surface and does not present a potential surface fault rupture hazard. However, the San Joaquin Hills Thrust is an active feature that can generate future earthquakes. The California Geological Survey (2003) considers this fault to be a separate potential seismic source and estimate an average slip rate of 0.5 mm/yr and a maximum magnitude of 6.6 for the San Joaquin Hills Thrust.

Northridge Thrust

The Northridge Thrust, as defined by Petersen et al. (1996), is an inferred deep thrust fault that is considered the eastern extension of the Oak Ridge fault. The Northridge Thrust is located beneath the majority of the San Fernando Valley and is believed to be the causative fault of the January 17, 1994 Northridge earthquake. This thrust fault is not exposed at the surface and does not present a potential surface fault rupture hazard. However, the Northridge Thrust is an active feature that can generate future earthquakes. The vertical surface projection of the Northridge Thrust is approximately 45 kilometers northwest of the site at the closest point. The California Geological Survey (2003) estimates an average slip rate of 1.5 mm/yr. and a maximum magnitude of 7.0 for the Northridge Thrust.

Potentially Active Faults

Los Alamitos Fault

The closest potentially active fault to the site is the Los Alamitos fault located approximately 6.9 kilometers to the northeast. This fault trends northwest-southeast from the northern boundary of the City of Lakewood, southeastward to the Los Alamitos Armed Forces Reserve Center. The fault, considered a southeasterly extension of the Paramount Syncline, appears to be a vertical fault with the early Pleistocene age materials on the west side of the fault displaced up relative to the east side. There is no evidence that this fault has offset Holocene age alluvial deposits (Ziony and Jones, 1989). Additionally, the "Fault Activity Map of California" published by the California Geological Survey (Jennings, 1994) depicts this fault to be potentially active.

Norwalk Fault

The potentially active Norwalk fault is located about 15 kilometers northeast of the site. The fault is a known ground-water barrier along the southern edge of the Coyote Hills, trending southeasterly toward the Santa Ana Mountains. The fault is thought to be a north-dipping reverse oblique fault along which the Coyote Hills have been uplifted. This fault offsets lower Pleistocene age and older deposits near the mouth of the Santa Ana Canyon. However, there is no evidence that this fault has offset Holocene age alluvial deposits (Ziony and Jones, 1989). Additionally, the State Geologist considers the Norwalk fault to be potentially active (Jennings, 1994).

6.5 GEOLOGIC-SEISMIC HAZARDS

Fault Rupture

The site is not within a currently established Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards. The closest Alquist-Priolo Earthquake Fault Zone, established for the active Cherry Hill fault of the Newport-Inglewood fault zone, is located approximately 0.9 kilometer (300 feet) northeast of the site. Based on the available geologic data, active or potentially active faults with the potential for surface fault rupture are not known to be located directly beneath or projecting toward the site. Therefore, the potential for surface rupture due to fault plane displacement propagating to the surface at the site during the design life of the project is considered low.

Seismicity

Earthquake Catalog Data

The seismicity of the region surrounding the site was determined from research of an electronic database of seismic data (Southern California Seismographic Network, 2004). This database includes earthquake data compiled by the California Institute of Technology for 1932 through 2003 and data for 1812 through 1931 compiled by Richter and the U.S. National Oceanic Atmospheric Administration (NOAA). The search for earthquakes that occurred within 100 kilometers of the site indicates that 411 earthquakes of Richter magnitude 4.0 and greater occurred from 1932 through 2003; 2 earthquakes of magnitude 6.0 or greater occurred between 1906 and 1931; and 1 earthquake of magnitude 7.0 or greater occurred between 1812 and 1905. A list of these earthquakes is presented as Table 3. Epicenters of moderate and major earthquakes (greater than magnitude 6.0) are shown in Figure 6.

The information for each earthquake includes date and time in Greenwich Civil Time (GCT), location of the epicenter in latitude and longitude, quality of epicentral determination (Q), depth in kilometers, distance from the site in kilometers, and magnitude. Where a depth of 0.0 is given, the solution was based on an assumed 16-kilometer focal depth. The explanation of the letter code for the quality factor of the data is presented on the first page of the table.

Historic Earthquakes

A number of earthquakes of moderate to major magnitude have occurred in the Southern California area within the last 70 years. A partial list of these earthquakes is included in the following table.

List of Historic Earthquakes

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Kilometers)	Direction to Epicenter
Long Beach	March 10, 1933	6.4	30	SE
Tehachapi	July 21, 1952	7.5	150	NW
San Fernando	February 9, 1971	6.6	69	NNW
Whittier Narrows	October 1, 1987	5.9	30	NNE
Sierra Madre	June 28, 1991	5.8	54	NNE
Landers	June 28, 1992	7.3	155	NE
Big Bear	June 28, 1992	6.4	127	NE
Northridge	January 17, 1994	6.7	55	NW
Hector Mine	October 16, 1999	7.1	195	NE

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be mitigated by proper engineering design and construction in conformance with current building codes and engineering practices.

Slope Stability

The relatively flat-lying topography at the site precludes both stability problems and the potential for lurching (earth movement at right angles to a cliff or steep slope during ground shaking). According to the City of Long Beach Seismic Safety Element (1988) and the County of Los Angeles Seismic Safety Element (1990), the site is not within an area identified as having a potential for slope instability. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Additionally, the site is not located within an area identified as having a potential for seismic slope instability (California Division of Mines and Geology, 1999).

Liquefaction and Seismic-Induced Settlement

Liquefaction potential is greatest where the ground water level is shallow, and submerged loose, fine sands occur within a depth of about 15 meters (50 feet) or less. Liquefaction potential decreases as grain size and clay and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases.

According to the City of Long Beach Seismic Safety Element (1988), and the County of Los Angeles Seismic Safety Element (1990), the site is not within an area identified as having a potential for liquefaction. Also, as shown in Figure 7, the site is not within a liquefaction hazard area as designated by the California Geological Survey (California Division of Mines and Geology, 1999). Based on the results of the SPT tests performed in our current explorations at the site and the depth to ground water, the potential for liquefaction in the submerged granular natural soils beneath the proposed building is low.

Seismic-induced settlement is often caused by loose to medium dense granular soils densified during ground shaking. Uniform settlement beneath a given structure would cause minimal damage. However, because of variations in distribution, density, and confining conditions of the soils, seismic-induced settlement is generally non-uniform and can cause serious structural damage. Dry and partially saturated soils as well as saturated granular soils are subject to seismic-induced settlement. Ground-water levels are not expected to rise above historic high water levels. Therefore, based on depth of the current and historic water levels and the dense nature of the materials beneath the site, the potential for liquefaction and the associated seismic-induced settlement at the site is considered to be low.

Tsunamis, Inundation, Seiches, and Flooding

The site located about 5 kilometers north of the Pacific Ocean at an elevation of approximately 14.6 to 14.9 meters (48 to 49 feet) above mean sea level (U. S. Geological Survey datum). According to the City of Long Beach Seismic Safety Element (1988) and County of Los Angeles Seismic Safety Element (1990), the site is not located within a tsunami run up zone. Therefore, due to the site elevation, the distance of the site from the ocean, and the fact that the site is not located within an identified tsunami run up zone, tsunamis (seismic sea waves) are not considered a significant hazard at the site.

According to the City of Long Beach Seismic Safety Element (1988) and the County of Los Angeles Seismic Safety Element (1990), the site is not located downslope of any large bodies of water that could adversely affect the site in the event of earthquake-induced dam failures or seiches (wave oscillations in an enclosed or semi-enclosed body of water).

The site is in an area of minimal flooding potential (Zone C) as defined by the Federal Insurance Administration.

Subsidence

The site is located at the edge of an area of known subsidence associated with fluid withdrawal during petroleum production. Subsidence in the Long Beach area as a result of oil production is well documented and was noted as early as the 1920's. Surveys conducted within the Long Beach subsidence area revealed an elliptical zone with up to 8.8 meters (29 feet) of subsidence at its center by 1970 (City of Long Beach, Department of Oil Properties, 1971). A lobe of lesser magnitude subsidence, centered on Signal Hill, is evident in maps published in 1970. According to contours of subsidence published by City of Long Beach, Department of Oil Properties (1971), the area of the site has undergone up to 0.6 meter (2 feet) of subsidence during the 1928 to 1970 monitoring period. However, water injection and repressurization programs in the Long Beach subsidence area have halted further subsidence and regional subsidence related to fluid withdrawal is not anticipated to adversely affect the site in the future.

The site is not located within an area of known subsidence associated with peat oxidation or hydrocompaction.

Oil Wells and Methane Gas

According to maps published by the California Division of Oil and Gas (CDOG, 1996), the site is located within the Long Beach Oil Field and there are several abandoned oil wells located within 0.3 kilometer (950 feet) of the site. According to CDOG records, these wells were abandoned in accordance with the requirements in effect during the time of their abandonment and may not meet the current CDOG standards of abandonment. There is a potential that these documented abandoned wells or other undocumented wells could be encountered during the proposed development at the site. Any wells encountered during construction will have to be abandoned in accordance with current CDOG standards and regulations. Additionally, since the site is within the limits of an oil field and there are wells nearby, there is a potential for methane or other volatile gases to be present at the site.

6.6 CONCLUSIONS

Based on the available geologic data, active or potentially active faults with the potential for surface fault rupture are not known to be located beneath or projecting toward the site. In our opinion, the potential for surface rupture at the site due to fault plane displacement propagating to the ground surface during the design life of the project is considered low. Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in Southern California and the effects of ground shaking can be mitigated by proper engineering design and construction in conformance with current building codes and engineering practices.

The site is not located within an area identified as having a potential for liquefaction and the depth to ground water beneath the site is greater than 15 meters (50 feet). Therefore, the potential for liquefaction at the site is low. The site is relatively level and the absence of nearby slopes precludes slope stability hazards. The potential for other geologic hazards such as tsunamis, inundation, seiches, flooding, and subsidence affecting the site is considered low.

As previously discussed, the site is located within the Long Beach Oil Field and there is a potential for methane and other volatile gases to occur at the site. Also, abandoned oil wells are documented within 0.3 kilometer (950 feet) of the site. Therefore, there is a potential that abandoned oil wells could be encountered during the proposed development at the site.

7.0 GROUND MOTION STUDY

Ground motions were postulated corresponding to earthquake levels having a 10% probability of exceedence during a 50-year time period (designated the Design Basis Earthquake, DBE) and a 10% probability of exceedence during in a 100-year time period (designated the Upper Bound Earthquake, UBE). The probabilistic response spectra developed for this study are referred to as the site-specific response spectra.

The site-specific response spectra for the DBE and UBE levels of shaking specified were determined by a Probabilistic Seismic Hazard Analysis (PSHA) using the computer program EZFRISK, Version 6.21. EZFRISK converts the slip rate of each fault into an activity rate using an algorithm consistent with the Anderson and Luco Occurrence Relation 2 (Anderson and Luco, 1983). The faults used in the study are shown in Tables 1 and 2, along with the maximum magnitude and the slip rate assigned to each fault.

The response spectra were developed using the average of the attenuation relations discussed in Abrahamson and Silva (1997), Boore et al. (1997), and Sadigh, et al. (1997), for a "soil" site type and a shear wave velocity in the upper 30 meters based on an appropriate value for an S_c soil type as defined by UBC 1997. The attenuations were modified for periods beyond 0.60 seconds to account for near source directivity effects as described in Sommerville, et al. (1997).

Dispersion in the ground motion attenuation relationships was considered by inclusion of the standard deviation of the ground motion data in the attenuation relationships used in the PSHA. We have used the relationships for rupture area versus magnitude of Wells and Coppersmith (1994) for the faults in our model. The response spectra for the horizontal component of shaking for the DBE and UBE ground motions are shown on Figures 8.1 and 8.2 for structural damping values of 2%, 5% and 10%. The response spectra in digitized form are presented in Tables 4 and 5. The estimated PGAs for the DBE and the UBE are 0.37g and 0.49g, respectively.

8.0 RECOMMENDATIONS

The existing fill soils are not considered suitable for support of the proposed building or floor slab. The natural soils at the site are generally firm and dense. The building can be supported on spread footings established in the firm and dense undisturbed natural soils. If the recommendations on grading are followed, the floor slab can be supported on grade. As an alternative, a mat-type foundation could be used as a foundation support.

8.1 FOUNDATIONS

Spread Footings

<u>Bearing Value</u>

Spread footings for the Todd Cancer Center (including the LAU) carried at least 1 foot into the undisturbed natural soils and at least 2 feet below the lowest adjacent grade or floor level can be designed to impose a net dead-plus-live load pressure of 5,000 pounds per square foot. The excavations should be deepened as necessary to extend into satisfactory natural soils.

A one-third increase in the bearing value can be used for wind or seismic loads. The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot; the weight of soil backfill can be neglected when determining the downward loads.

Minor structures can be supported on spread footings and footings for such structures that are structurally separate from the building can be designed to impose a net dead-plus-live load pressure of 1,000 pounds per square foot at a depth of 1½ feet below the lowest adjacent grade. Such footings can be established in either properly compacted fill soils or undisturbed natural soils.

Settlement

We estimate the settlement of the building, supported on spread footings in the manner recommended, will be about one inch. Differential settlement between adjacent columns is expected to be about ¼ inch.

Lateral Resistance

Lateral loads can be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.4 can be used between the structure footings and the floor slab and the supporting soils. The passive resistance of natural soils or properly compacted fill soils can be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. A one-third increase in the passive value can be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

Mat-type Foundation

Bearing Value

A mat-type foundation for the Todd Cancer Institute building (including the LAU) carried at least 1 foot into properly compacted fill soils and/or the undisturbed natural soils and at least 2 feet below the lowest adjacent grade or floor level can be designed to impose a net dead-plus-live load pressure of up to 1,000 pounds per square foot. The excavations should be deepened as necessary to extend into satisfactory soils.

A one-third increase can be used for wind or seismic loads. The recommended bearing value is a net value, and the weight of concrete in the mat foundation can be taken as 50 pounds per cubic foot; the weight of soil backfill can be neglected when determining the downward loads.

Settlement

We estimate the settlement of the building, supported on a mat-type foundation in the manner recommended and imposing a soils pressure of about 600 pounds per square foot, will be about one inch.

Lateral Resistance

Lateral loads can be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.4 can be used between the structure foundation and the supporting soils. The passive resistance of natural soils or properly compacted fill soils against the mat foundation can be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. A one-third increase in the passive value can be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

Modulus of Subgrade Reaction

For structural analyses of spread footings or mat-type foundations, a vertical modulus of subgrade reaction of 300 pounds per cubic inch may be used. This value is a unit value for use with a 1-foot-square area. The modulus should be reduced in accordance with the following equation when used with the larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

where K = unit subgrade modulus
K_R = reduced subgrade modulus
B = spread foundation width

8.2 SITE COEFFICIENT AND SEISMIC ZONATION

The site coefficient, S, can be determined as established in the Earthquake Regulations under Section 1629 of the California Building Code (CBC), 2001 edition, for seismic design of the proposed Todd Cancer Institute building. Based on a review of the local soil and geologic conditions, the site may be classified as Soil Profile Type S_C, as specified in the 2001 code. The site is located within CBC Seismic Zone 4.

The site is near the Newport-Inglewood Fault, which has been determined to be a Type B seismic source by the California Division of Mines and Geology. According to Map M-33 in the 1998 publication from the International Conference of Building Officials entitled "Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada," the proposed building is located at a distance of less than 2 kilometers from the Newport-Inglewood Fault. At this distance for a seismic source type B, the near source factors, N_a and N_v, are to be taken as 1.3 and 1.6, respectively, based on Tables 16-S and 16-T of the 2001 CBC.

8.3 FLOOR SLAB SUPPORT

If the subgrade is prepared as recommended in the following section on grading, the building floor slab can be supported on grade. The on-site clayey soils are expansive, and where the clayey soils are encountered beneath the building floor slab and beneath concrete walks and slabs, the upper 2 feet of the clayey soils should be removed and replaced with properly compacted fill consisting of relatively non-expansive soils with an expansion index of less than 35.

Construction activities and exposure to the environment can cause deterioration of the prepared subgrade. Therefore, we recommend our that our field representative observe the condition of the final subgrade soils immediately prior to slab-on-grade construction, and, if necessary, perform further density and moisture content tests to determine the suitability of the final prepared subgrade.

If vinyl or other moisture-sensitive floor covering is planned, we recommend that the floor slab in those areas be underlain by a capillary break consisting of a vapor-retarding membrane over a 4-inch-thick layer of gravel. A 2-inch-thick layer of sand should be placed between the gravel and the membrane to decrease the possibility of damage to the membrane. We suggest the following gradation for the gravel:

Sieve Size	Percent Passing
¾"	90 - 100
No. 4	0 - 10
No. 100	0 - 3

A low-slump concrete should be used to reduce possible curling of the slab. A 2-inch-thick layer of coarse sand can be placed over the vapor retarding membrane to reduce slab curling. If this sand bedding is used, care should be taken during the placement of the concrete to prevent displacement of the sand. The concrete slab should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering. The sand and gravel layers can be considered part of the required non-expansive soil layer under the concrete slabs.

8.4 PAVING

To provide support for paving, the subgrade soils should be prepared as recommended in the following section on grading. Compaction of the subgrade, including trench backfills, to at least 90%, and achieving a firm, hard, and unyielding surface will be important for paving support. The preparation of the paving area subgrade should be done immediately prior to placement of the base course. Proper drainage of the paved areas should be provided since this will reduce moisture infiltration into the subgrade and increase the life of the paving.

To provide data for design of asphalt paving, the R-value of a sample of the upper soils was determined. The test results, which indicate an R-value of 51, are presented in the Appendix.

The required paving and base thicknesses will depend on the expected wheel loads and volume of traffic (Traffic Index or TI). Assuming that the paving subgrade will consist of the on-site or comparable soils compacted to at least 90% as recommended, the minimum recommended paving thicknesses are presented in the following table.

Paving Thickness

Traffic Use	Traffic Index	Asphaltic Concrete (inches)	Base Course (inches)
Automobile Parking	4	3	6
Driveways with Light Truck Traffic	6	4	6
Roadways with Heavy Truck Traffic	7	5	6

The asphalt paving sections were determined using the Caltrans Asphalt Institute design method. We can determine the recommended paving and base course thicknesses for other Traffic Indices if required. Careful inspection is recommended to evaluate that the recommended thicknesses or greater are achieved, and that proper construction procedures are followed.

The base course should conform to requirements of Section 26 of State of California Department of Transportation Standard Specifications (Caltrans), latest edition, or meet the specifications for untreated base as defined in Section 200-2 of the latest edition of the Standard Specifications for Public Works Construction (Green Book). The base course should be compacted to at least 95%.

Portland Cement Concrete Paving

Portland cement concrete paving sections were determined in accordance with procedures developed by the Portland Cement Association. Concrete paving sections for a range of Traffic Indices are presented below for pavement on soil with an R-value of 51. We have assumed that the portland cement concrete will have a compressive strength of at least 3,000 pounds per square inch.

Paving Thickness

Traffic Use	Traffic Index	Concrete Paving (inches)	Base Course (inches)
Automobile Parking	4	6	4
Driveways with Light Truck Traffic	6	6½	4
Roadways with Heavy Truck Traffic	7	7	4

We recommend that the concrete paving be properly reinforced. In addition, dowels are recommended at joints in the paving to reduce any possible offsets.

Base Course

The base course should conform to requirements of Section 26 of State of California Department of Transportation Standard Specifications (Caltrans), latest edition, or meet the specifications for untreated base as defined in Section 200-2 of the latest edition of the Standard Specifications for Public Works Construction (Green Book). The base course should be compacted to at least 95%.

8.5 GRADING

The existing fill soils are not uniformly well compacted and are not considered suitable for support of paving or floor slabs on grade. The existing fill soils should be excavated and replaced as properly compacted fill. All required fill should be uniformly well compacted and observed and tested during placement. The on-site soils can be used in any required fill.

This section gives recommendations for the following grading considerations:

- Site preparation (includes specifications for compaction of natural soils).
- Excavations and Temporary Slopes.
- Compaction (specifications for fill compaction).
- Material for fill (specifications for on-site and import materials).

Site Preparation

After the site is cleared, all the existing fill soils within the building area should be excavated. The excavation of the fill soils should extend at least 5 feet beyond the exterior footings of the buildings, where possible, and should extend beneath concrete walks and slabs, and beneath asphaltic and concrete paving. Next, the exposed natural soils should be carefully observed for the removal of all unsuitable deposits and the exposed natural soils should be scarified to a depth of 6 inches, brought to near-optimum moisture content, and rolled with heavy compaction equipment. At least the upper 6 inches of the exposed soils should be compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557-91 method of compaction.

The on-site clayey soils are expansive and where these soils are encountered beneath the building floor slab and beneath concrete walls and slabs, they should be removed to allow the placement of at least 2 feet of relatively non-expansive soil with an expansion under of less than 35. Good drainage of surface water should be provided by adequately sloping all surfaces. Such drainage will be important to reduce infiltration of water beneath floor slabs and pavement.

Excavations and Temporary Slopes

Where excavations are deeper than about 4 feet, the sides of the excavations should be sloped back at 1:1 (horizontal to vertical) or shored for safety. Unshored excavations should not extend below a plane drawn at 1½:1 (horizontal to vertical) extending downward from adjacent existing footings. We would be pleased to present data for design of shoring if required.

Excavations should be observed by personnel of our firm so that any necessary modifications based on variations in the soil conditions can be made. All applicable safety requirements and regulations, including OSHA regulations, should be met.

Compaction

Any required fill should be placed in loose lifts not more than 8-inches-thick and compacted. The fill should be compacted to at least 90% of the maximum density obtainable by the ASTM Designation D1557-91 method of compaction. The moisture content of the on-site soils at the time of compaction should vary no more than 2% below or above optimum moisture content. The moisture content of the on-site clayey soils at the time of compaction should be between 2% and 4% above optimum moisture content.

Backfill

All required backfill should be mechanically compacted in layers; flooding should not be permitted. Proper compaction of backfill will be necessary to reduce settlement of the backfill and to reduce settlement of overlying slabs and paving. Backfill should be compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557-91 method of compaction. The on-site soils can be used in the compacted backfill. However, the on-site soils are expansive and will be difficult to compact, and should not be used within the upper backfill. The on-site soils can be used in the upper 2 feet of the backfill, except beneath the floor slab and beneath concrete walks and slabs, to provide a relatively impermeable layer when compacted to restrict the inflow of surface water into the backfill. The exterior grades should be sloped to drain away from the foundations to prevent ponding of water.

Some settlement of the backfill should be expected, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the building. Also, provisions should be made for some settlement of concrete walks supported on backfill.

Material for Fill

The on-site soils, less any debris or organic matter within the fill soils, can be used in required fills. However, because of their expansive characteristics, the on-site clayey soils should not be used within 2 feet of the subgrade for floor slabs, walks, and other slabs. Cobbles larger than 3 inches in diameter should not be used in the fill. Any required import material should consist of relatively non-expansive soils with an expansion index of less than 35. The imported materials should contain sufficient fines (binder material) so as to be relatively impermeable and result in a stable subgrade when compacted. All proposed import materials should be approved by our personnel prior to being placed at the site.

8.6 GEOTECHNICAL OBSERVATION

The reworking of the upper soils and the compaction of all required fill should be observed and tested during placement by a representative of our firm. This representative should perform at least the following duties:

- Observe the clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade. The representative should also observe proofrolling and delineation of areas requiring overexcavation.
- Evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.
- Observe the fill and backfill for uniformity during placement.
- Test backfill for field density and compaction to determine the percentage of compaction achieved during backfill placement.
- Observe and probe foundation materials to confirm that suitable bearing materials are present at the design foundation depths.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits can be obtained and arrangements can be made for required inspection(s). The contractor should be familiar with the inspection requirements of the reviewing agencies.

9.0 BASIS FOR RECOMMENDATIONS

The recommendations provided in this report are based upon our understanding of the described project information and on our interpretation of the data collected during our current subsurface explorations. We have made our recommendations based upon experience with similar subsurface conditions under similar loading conditions. The recommendations apply to the specific project discussed in this report; therefore, any change in the structure configuration, loads, location, or the site grades should be provided to us so that we can review our conclusions and recommendations and make any necessary modifications.

The recommendations provided in this report are also based upon the assumption that the necessary geotechnical observations and testing during construction will be performed by representatives of our firm. The field observation services are considered a continuation of the geotechnical investigation and essential to verify that the actual soil conditions are as expected. This also provides for the procedure whereby the client can be advised of unexpected or changed conditions that would require modifications of our original recommendations. In addition, the presence of our representative at the site provides the client with an independent professional opinion regarding the geotechnically related construction procedures. If another firm is retained for the geotechnical observation services, our professional responsibility and liability would be limited to the extent that we would not be the geotechnical engineer of record.

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TABLES

Table 1
Major Named Faults Considered to be Active in Southern California

Fault (increasing distance)	Maximum Magnitude			Slip Rate (mm/yr.)	Distance From Site (kilometers)	Direction From Site
Newport-Inglewood Zone	7.1	(a)	SS	1.0	0.2	NE
Palos Verdes	7.3	(a)	SS	3.0	10½	SW
Puente Hills Blind Thrust	7.1	(a)	BT	0.7	11½	NE
Upper Elysian Park	6.4	(a)	BT	1.3	25	N
Whittier	6.8	(a)	SS	2.5	25	NE
San Joaquin Hills	6.6	(a)	BT	0.5	27	SW
Santa Monica	6.6	(a)	RO	1.0	33	NW
Raymond	6.5	(a)	RO	1.5	34	N
Hollywood	6.4	(a)	RO	1.0	36	N
Verdugo	6.9	(a)	RO	0.5	38	N
Malibu Coast	6.7	(a)	RO	0.3	41	NW
Sierra Madre	7.2	(a)	RO	2.0	42	NE
Northridge Thrust	7.0	(a)	BT	1.5	45	NW
Chino - Central Avenue	6.7	(e)	NO	1.0	49	NE
Elsinore (Glen Ivy Segment)	6.8	(a)	SS	5.0	49	E
San Gabriel	7.2	(a)	SS	1.0	51	NNE
Anacapa-Dume	7.5	(a)	RO	3.0	52	NW
San Fernando	6.7	(a)	RO	2.0	52	N
Cucamonga	6.9	(a)	RO	5.0	57	NE
Simi-Santa Rosa	7.0	(a)	RO	1.0	72	NW
San Andreas (Mojave Segment)	7.4	(a)	SS	30.0	78	NE
Oak Ridge	7.0	(a)	RO	4.0	79	NW
San Jacinto (San Bernardino Segment)	6.7	(a)	SS	12.0	79	NE
San Cayetano	7.0	(a)	RO	6.0	100	NW

(a) California Geological Survey, 2003
(b) Mark, 1977
(c) Slemmons, 1979
(d) Wesnousky, 1986
(e) Hummon et al., 1994
SS Strike Slip
NO Normal Oblique
RO Reverse Oblique
BT Blind Thrust

Table 2
Major Named Faults Considered to be Potentially Active in Southern California

Fault (increasing distance)	Maximum Magnitude			Slip Rate (mm/yr.)	Distance From Site (kilometers)	Direction From Site
Los Alamitos	6.2	(b)	SS	0.1	6.9	NE
Norwalk	6.7	(c)	RO	0.1	15	NE
Charnock	6.5	(c)	SS	0.1	25	NW
Coyote Pass	6.7	(b)	RO	0.1	25	N
Overland	6.0	(c)	SS	0.1	26	NW
MacArthur Park	5.7	(e)	RO	0.1	27	N
Clamshell-Sawpit	6.5	(a)	RO	0.5	43	NE
Duarte	6.7	(c)	RO	0.1	43	NNE
San Jose	6.4	(a)	RO	0.5	44	NE
Indian Hill	6.6	(b)	RO	0.1	45	NE
Northridge Hills	6.6	(d)	SS	1.2	53	NW
Santa Susana	6.7	(a)	RO	5.0	63	NW
Holser	6.5	(a)	RO	0.4	81	NW

(a) California Geological Survey, 2003
(b) Mark, 1977
(c) Slemmons, 1979
(d) Wesnousky, 1986
(e) Hummon et al., 1994
SS Strike Slip
NO Normal Oblique
RO Reverse Oblique
BT Blind Thrust

Table 3
List of Historic Earthquakes Of Magnitude 4.0 or Greater Within 100 Km Of The Site
(CAL TECH DATA 1932-2003)

DATE	TIME	LATITUDE		LONGITUDE		Q	DIST	DEPTH	MAGNITUDE
11-01-1932	04:45:00	34.00	N	117.25	W	E	89	.0	4.0
03-11-1933	01:54:07	33.62	N	117.97	W	A	30	.0	6.4
03-11-1933	02:04:00	33.75	N	118.08	W	C	12	.0	4.9
03-11-1933	02:05:00	33.75	N	118.08	W	C	12	.0	4.3
03-11-1933	02:09:00	33.75	N	118.08	W	C	12	.0	5.0
03-11-1933	02:10:00	33.75	N	118.08	W	C	12	.0	4.6
03-11-1933	02:11:00	33.75	N	118.08	W	C	12	.0	4.4
03-11-1933	02:16:00	33.75	N	118.08	W	C	12	.0	4.8
03-11-1933	02:17:00	33.60	N	118.00	W	E	29	.0	4.5
03-11-1933	02:22:00	33.75	N	118.08	W	C	12	.0	4.0
03-11-1933	02:27:00	33.75	N	118.08	W	C	12	.0	4.6
03-11-1933	02:30:00	33.75	N	118.08	W	C	12	.0	5.1
03-11-1933	02:31:00	33.60	N	118.00	W	E	29	.0	4.4
03-11-1933	02:52:00	33.75	N	118.08	W	C	12	.0	4.0
03-11-1933	02:57:00	33.75	N	118.08	W	C	12	.0	4.2
03-11-1933	02:58:00	33.75	N	118.08	W	C	12	.0	4.0
03-11-1933	02:59:00	33.75	N	118.08	W	C	12	.0	4.6
03-11-1933	03:05:00	33.75	N	118.08	W	C	12	.0	4.2
03-11-1933	03:09:00	33.75	N	118.08	W	C	12	.0	4.4
03-11-1933	03:11:00	33.75	N	118.08	W	C	12	.0	4.2
03-11-1933	03:23:00	33.75	N	118.08	W	C	12	.0	5.0
03-11-1933	03:36:00	33.75	N	118.08	W	C	12	.0	4.0
03-11-1933	03:39:00	33.75	N	118.08	W	C	12	.0	4.0
03-11-1933	03:47:00	33.75	N	118.08	W	C	12	.0	4.1
03-11-1933	04:36:00	33.75	N	118.08	W	C	12	.0	4.6
03-11-1933	04:39:00	33.75	N	118.08	W	C	12	.0	4.9
03-11-1933	04:40:00	33.75	N	118.08	W	C	12	.0	4.7
03-11-1933	05:10:22	33.70	N	118.07	W	C	17	.0	5.1
03-11-1933	05:13:00	33.75	N	118.08	W	C	12	.0	4.7
03-11-1933	05:15:00	33.75	N	118.08	W	C	12	.0	4.0
03-11-1933	05:18:04	33.58	N	117.98	W	C	32	.0	5.2
03-11-1933	05:21:00	33.75	N	118.08	W	C	12	.0	4.4
03-11-1933	05:24:00	33.75	N	118.08	W	C	12	.0	4.2
03-11-1933	05:53:00	33.75	N	118.08	W	C	12	.0	4.0
03-11-1933	05:55:00	33.75	N	118.08	W	C	12	.0	4.0

NOTE: Q IS A FACTOR RELATING THE QUALITY OF EPICENTRAL DETERMINATION

A = +- 1 km horizontal distance; +- 2 km depth
B = +- 2 km horizontal distance; +- 5 km depth
C = +- 5 km horizontal distance; no depth restriction
D = >+- 5 km horizontal distance

Event qualities are highly suspect prior to 1990. Many of these event qualities are based on incomplete information according to Caltech.

Table 3
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(CAL TECH DATA 1932-2003)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
03-11-1933	06:11:00	33.75 N	118.08 W	C	12	.0	4.4
03-11-1933	06:18:00	33.75 N	118.08 W	C	12	.0	4.2
03-11-1933	06:29:00	33.85 N	118.27 W	C	8	.0	4.4
03-11-1933	06:35:00	33.75 N	118.08 W	C	12	.0	4.2
03-11-1933	06:58:03	33.68 N	118.05 W	C	19	.0	5.5
03-11-1933	07:51:00	33.75 N	118.08 W	C	12	.0	4.2
03-11-1933	07:59:00	33.75 N	118.08 W	C	12	.0	4.1
03-11-1933	08:08:00	33.75 N	118.08 W	C	12	.0	4.5
03-11-1933	08:32:00	33.75 N	118.08 W	C	12	.0	4.2
03-11-1933	08:37:00	33.75 N	118.08 W	C	12	.0	4.0
03-11-1933	08:54:57	33.70 N	118.07 W	C	17	.0	5.1
03-11-1933	09:10:00	33.75 N	118.08 W	C	12	.0	5.1
03-11-1933	09:11:00	33.75 N	118.08 W	C	12	.0	4.4
03-11-1933	09:26:00	33.75 N	118.08 W	C	12	.0	4.1
03-11-1933	10:25:00	33.75 N	118.08 W	C	12	.0	4.0
03-11-1933	10:45:00	33.75 N	118.08 W	C	12	.0	4.0
03-11-1933	11:00:00	33.75 N	118.08 W	C	12	.0	4.0
03-11-1933	11:04:00	33.75 N	118.13 W	C	9	.0	4.6
03-11-1933	11:29:00	33.75 N	118.08 W	C	12	.0	4.0
03-11-1933	11:38:00	33.75 N	118.08 W	C	12	.0	4.0
03-11-1933	11:41:00	33.75 N	118.08 W	C	12	.0	4.2
03-11-1933	11:47:00	33.75 N	118.08 W	C	12	.0	4.4
03-11-1933	12:50:00	33.68 N	118.05 W	C	19	.0	4.4
03-11-1933	13:50:00	33.73 N	118.10 W	C	12	.0	4.4
03-11-1933	13:57:00	33.75 N	118.08 W	C	12	.0	4.0
03-11-1933	14:25:00	33.85 N	118.27 W	C	8	.0	5.0
03-11-1933	14:47:00	33.73 N	118.10 W	C	12	.0	4.4
03-11-1933	14:57:00	33.88 N	118.32 W	C	14	.0	4.9
03-11-1933	15:09:00	33.73 N	118.10 W	C	12	.0	4.4
03-11-1933	15:47:00	33.75 N	118.08 W	C	12	.0	4.0
03-11-1933	16:53:00	33.75 N	118.08 W	C	12	.0	4.8
03-11-1933	19:44:00	33.75 N	118.08 W	C	12	.0	4.0
03-11-1933	19:56:00	33.75 N	118.08 W	C	12	.0	4.2
03-11-1933	22:00:00	33.75 N	118.08 W	C	12	.0	4.4
03-11-1933	22:31:00	33.75 N	118.08 W	C	12	.0	4.4

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DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
03-11-1933	22:32:00	33.75 N	118.08 W	C	12	.0	4.1
03-11-1933	22:40:00	33.75 N	118.08 W	C	12	.0	4.4
03-11-1933	23:05:00	33.75 N	118.08 W	C	12	.0	4.2
03-12-1933	00:27:00	33.75 N	118.08 W	C	12	.0	4.4
03-12-1933	00:34:00	33.75 N	118.08 W	C	12	.0	4.0
03-12-1933	04:48:00	33.75 N	118.08 W	C	12	.0	4.0
03-12-1933	05:46:00	33.75 N	118.08 W	C	12	.0	4.4
03-12-1933	06:01:00	33.75 N	118.08 W	C	12	.0	4.2
03-12-1933	06:16:00	33.75 N	118.08 W	C	12	.0	4.6
03-12-1933	07:40:00	33.75 N	118.08 W	C	12	.0	4.2
03-12-1933	08:35:00	33.75 N	118.08 W	C	12	.0	4.2
03-12-1933	15:02:00	33.75 N	118.08 W	C	12	.0	4.2
03-12-1933	16:51:00	33.75 N	118.08 W	C	12	.0	4.0
03-12-1933	17:38:00	33.75 N	118.08 W	C	12	.0	4.5
03-12-1933	18:25:00	33.75 N	118.08 W	C	12	.0	4.1
03-12-1933	21:28:00	33.75 N	118.08 W	C	12	.0	4.1
03-12-1933	23:54:00	33.75 N	118.08 W	C	12	.0	4.5
03-13-1933	03:43:00	33.75 N	118.08 W	C	12	.0	4.1
03-13-1933	04:32:00	33.75 N	118.08 W	C	12	.0	4.7
03-13-1933	06:17:00	33.75 N	118.08 W	C	12	.0	4.0
03-13-1933	13:18:28	33.75 N	118.08 W	C	12	.0	5.3
03-13-1933	15:32:00	33.75 N	118.08 W	C	12	.0	4.1
03-13-1933	19:29:00	33.75 N	118.08 W	C	12	.0	4.2
03-14-1933	00:36:00	33.75 N	118.08 W	C	12	.0	4.2
03-14-1933	12:19:00	33.75 N	118.08 W	C	12	.0	4.5
03-14-1933	19:01:50	33.62 N	118.02 W	C	27	.0	5.1
03-14-1933	22:42:00	33.75 N	118.08 W	C	12	.0	4.1
03-15-1933	02:08:00	33.75 N	118.08 W	C	12	.0	4.1
03-15-1933	04:32:00	33.75 N	118.08 W	C	12	.0	4.1
03-15-1933	05:40:00	33.75 N	118.08 W	C	12	.0	4.2
03-15-1933	11:13:32	33.62 N	118.02 W	C	27	.0	4.9
03-16-1933	14:56:00	33.75 N	118.08 W	C	12	.0	4.0
03-16-1933	15:29:00	33.75 N	118.08 W	C	12	.0	4.2
03-16-1933	15:30:00	33.75 N	118.08 W	C	12	.0	4.1
03-17-1933	16:51:00	33.75 N	118.08 W	C	12	.0	4.1

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DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
03-18-1933	20:52:00	33.75 N	118.08 W	C	12	.0	4.2
03-19-1933	21:23:00	33.75 N	118.08 W	C	12	.0	4.2
03-20-1933	13:58:00	33.75 N	118.08 W	C	12	.0	4.1
03-21-1933	03:26:00	33.75 N	118.08 W	C	12	.0	4.1
03-23-1933	08:40:00	33.75 N	118.08 W	C	12	.0	4.1
03-23-1933	18:31:00	33.75 N	118.08 W	C	12	.0	4.1
03-25-1933	13:46:00	33.75 N	118.08 W	C	12	.0	4.1
03-30-1933	12:25:00	33.75 N	118.08 W	C	12	.0	4.4
03-31-1933	10:49:00	33.75 N	118.08 W	C	12	.0	4.1
04-01-1933	06:42:00	33.75 N	118.08 W	C	12	.0	4.2
04-02-1933	08:00:00	33.75 N	118.08 W	C	12	.0	4.0
04-02-1933	15:36:00	33.75 N	118.08 W	C	12	.0	4.0
05-16-1933	20:58:55	33.75 N	118.17 W	C	7	.0	4.0
08-04-1933	04:17:48	33.75 N	118.18 W	C	7	.0	4.0
10-02-1933	09:10:17	33.78 N	118.13 W	A	6	.0	5.4
10-02-1933	13:26:01	33.62 N	118.02 W	C	27	.0	4.0
10-25-1933	07:00:46	33.95 N	118.13 W	C	16	.0	4.3
11-13-1933	21:28:00	33.87 N	118.20 W	C	6	.0	4.0
11-20-1933	10:32:00	33.78 N	118.13 W	B	6	.0	4.0
01-09-1934	14:10:00	34.10 N	117.68 W	A	57	.0	4.5
01-18-1934	02:14:00	34.10 N	117.68 W	A	57	.0	4.0
01-20-1934	21:17:00	33.62 N	118.12 W	B	23	.0	4.5
04-17-1934	18:33:00	33.57 N	117.98 W	C	33	.0	4.0
10-17-1934	09:38:00	33.63 N	118.40 W	B	28	.0	4.0
11-16-1934	21:26:00	33.75 N	118.00 W	B	19	.0	4.0
06-19-1935	11:17:00	33.72 N	117.52 W	B	63	.0	4.0
07-13-1935	10:54:16	34.20 N	117.90 W	A	51	.0	4.7
09-03-1935	06:47:00	34.03 N	117.32 W	B	84	.0	4.5
12-25-1935	17:15:00	33.60 N	118.02 W	B	28	.0	4.5
02-23-1936	22:20:42	34.13 N	117.34 W	A	86	10.0	4.5
02-26-1936	09:33:27	34.14 N	117.34 W	A	87	10.0	4.0
08-22-1936	05:21:00	33.77 N	117.82 W	B	35	.0	4.0
10-29-1936	22:35:36	34.38 N	118.62 W	C	75	10.0	4.0
01-15-1937	18:35:47	33.56 N	118.06 W	B	30	10.0	4.0
03-19-1937	01:23:38	34.11 N	117.43 W	A	78	10.0	4.0

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(CAL TECH DATA 1932-2003)

DATE	TIME	LATITUDE		LONGITUDE		Q	DIST	DEPTH	MAGNITUDE
07-07-1937	11:12:00	33.57	N	117.98	W	B	33	.0	4.0
09-01-1937	13:48:08	34.21	N	117.53	W	A	75	10.0	4.5
09-01-1937	16:35:33	34.18	N	117.55	W	A	72	10.0	4.5
09-13-1937	22:14:39	33.04	N	118.73	W	C	100	10.0	4.0
05-21-1938	09:44:00	33.62	N	118.03	W	B	26	.0	4.0
05-31-1938	08:34:55	33.70	N	117.51	W	B	64	10.0	5.2
07-05-1938	18:06:55	33.68	N	117.55	W	A	61	10.0	4.5
08-06-1938	22:00:55	33.72	N	117.51	W	B	64	10.0	4.0
08-31-1938	03:18:14	33.76	N	118.25	W	A	8	10.0	4.5
11-29-1938	19:21:15	33.90	N	118.43	W	A	25	10.0	4.0
12-07-1938	03:38:00	34.00	N	118.42	W	B	30	.0	4.0
12-27-1938	10:09:28	34.13	N	117.52	W	B	71	10.0	4.0
04-03-1939	02:50:44	34.04	N	117.23	W	A	93	10.0	4.0
11-04-1939	21:41:00	33.77	N	118.12	W	B	8	.0	4.0
11-07-1939	18:52:08	34.00	N	117.28	W	A	86	.0	4.7
12-27-1939	19:28:49	33.78	N	118.20	W	A	3	.0	4.7
01-13-1940	07:49:07	33.78	N	118.13	W	B	6	.0	4.0
02-08-1940	16:56:17	33.70	N	118.07	W	B	17	.0	4.0
02-11-1940	19:24:10	33.98	N	118.30	W	B	22	.0	4.0
04-18-1940	18:43:43	34.03	N	117.35	W	A	81	.0	4.4
06-05-1940	08:27:27	33.83	N	117.40	W	B	73	.0	4.0
07-20-1940	04:01:13	33.70	N	118.07	W	B	17	.0	4.0
10-11-1940	05:57:12	33.77	N	118.45	W	A	25	.0	4.7
10-12-1940	00:24:00	33.78	N	118.42	W	B	21	.0	4.0
10-14-1940	20:51:11	33.78	N	118.42	W	B	21	.0	4.0
11-01-1940	07:25:03	33.78	N	118.42	W	B	21	.0	4.0
11-01-1940	20:00:46	33.63	N	118.20	W	B	20	.0	4.0
11-02-1940	02:58:26	33.78	N	118.42	W	B	21	.0	4.0
01-30-1941	01:34:46	33.97	N	118.05	W	A	22	.0	4.1
03-22-1941	08:22:40	33.52	N	118.10	W	B	34	.0	4.0
03-25-1941	23:43:41	34.22	N	117.47	W	B	81	.0	4.0
04-11-1941	01:20:24	33.95	N	117.58	W	B	58	.0	4.0
10-22-1941	06:57:18	33.82	N	118.22	W	A	3	.0	4.8
11-14-1941	08:41:36	33.78	N	118.25	W	A	6	.0	4.8
04-16-1942	07:28:33	33.37	N	118.15	W	C	49	.0	4.0

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Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(CAL TECH DATA 1932-2003)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
10-24-1943	00:29:21	33.93 N	117.37 W	C	77	.0	4.0
06-19-1944	00:03:33	33.87 N	118.22 W	B	7	.0	4.5
06-19-1944	03:06:07	33.87 N	118.22 W	C	7	.0	4.4
02-24-1946	06:07:52	34.40 N	117.80 W	C	75	.0	4.1
06-01-1946	11:06:31	34.42 N	118.83 W	C	90	.0	4.1
03-01-1948	08:12:13	34.17 N	117.53 W	B	72	.0	4.7
04-16-1948	22:26:24	34.02 N	118.97 W	B	76	.0	4.7
10-03-1948	02:46:28	34.18 N	117.58 W	A	70	.0	4.0
01-11-1950	21:41:35	33.94 N	118.20 W	A	14	.4	4.1
09-22-1951	08:22:39	34.12 N	117.34 W	A	86	11.9	4.3
02-10-1952	13:50:55	33.58 N	119.18 W	C	95	.0	4.0
02-17-1952	12:36:58	34.00 N	117.27 W	A	88	16.0	4.5
08-23-1952	10:09:07	34.52 N	118.20 W	A	79	13.1	5.1
10-26-1954	16:22:26	33.73 N	117.47 W	B	67	.0	4.1
05-15-1955	17:03:25	34.12 N	117.48 W	A	74	7.6	4.0
05-29-1955	16:43:35	33.99 N	119.06 W	B	83	17.4	4.1
01-03-1956	00:25:48	33.72 N	117.50 W	B	65	13.7	4.7
02-07-1956	02:16:56	34.53 N	118.64 W	B	90	16.0	4.2
02-07-1956	03:16:38	34.59 N	118.61 W	A	95	2.6	4.6
03-25-1956	03:32:02	33.60 N	119.11 W	A	88	8.2	4.2
06-28-1960	20:00:48	34.12 N	117.47 W	A	74	12.0	4.1
10-04-1961	02:21:31	33.85 N	117.75 W	B	41	4.3	4.1
10-20-1961	19:49:50	33.65 N	117.99 W	B	25	4.6	4.3
10-20-1961	20:07:14	33.66 N	117.98 W	B	26	6.1	4.0
10-20-1961	21:42:40	33.67 N	117.98 W	B	25	7.2	4.0
10-20-1961	22:35:34	33.67 N	118.01 W	B	23	5.6	4.1
11-20-1961	08:53:34	33.68 N	117.99 W	B	23	4.4	4.0
04-27-1962	09:12:32	33.74 N	117.19 W	B	93	5.7	4.1
09-14-1963	03:51:16	33.54 N	118.34 W	B	33	2.2	4.2
08-30-1964	22:57:37	34.27 N	118.44 W	B	56	15.4	4.0
01-01-1965	08:04:18	34.14 N	117.52 W	B	72	5.9	4.4
04-15-1965	20:08:33	34.13 N	117.43 W	B	79	5.5	4.5
07-16-1965	07:46:22	34.49 N	118.52 W	B	81	15.1	4.0
01-08-1967	07:37:30	33.63 N	118.47 W	B	33	11.4	4.0
01-08-1967	07:38:05	33.66 N	118.41 W	C	26	17.7	4.0

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Table 3
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(CAL TECH DATA 1932-2003)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
06-15-1967	04:58:05	34.00 N	117.97 W	B	29	10.0	4.1
02-28-1969	04:56:12	34.57 N	118.11 W	A	84	5.3	4.3
05-05-1969	16:02:09	34.30 N	117.57 W	B	79	8.8	4.4
10-24-1969	20:26:42	33.34 N	119.10 W	B	100	-1.8	4.7
10-27-1969	13:16:02	33.55 N	117.81 W	B	46	6.5	4.5
10-31-1969	10:39:28	33.43 N	119.10 W	B	94	7.3	4.7
09-12-1970	14:10:11	34.27 N	117.52 W	A	80	8.0	4.1
09-12-1970	14:30:52	34.27 N	117.54 W	A	79	8.0	5.2
09-13-1970	04:47:48	34.28 N	117.55 W	A	79	8.0	4.4
02-09-1971	14:00:41	34.41 N	118.40 W	B	69	8.4	6.6
02-09-1971	14:01:08	34.41 N	118.40 W	D	69	8.0	5.8
02-09-1971	14:01:33	34.41 N	118.40 W	D	69	8.0	4.2
02-09-1971	14:01:40	34.41 N	118.40 W	D	69	8.0	4.1
02-09-1971	14:01:50	34.41 N	118.40 W	D	69	8.0	4.5
02-09-1971	14:01:54	34.41 N	118.40 W	D	69	8.0	4.2
02-09-1971	14:01:59	34.41 N	118.40 W	D	69	8.0	4.1
02-09-1971	14:02:03	34.41 N	118.40 W	D	69	8.0	4.1
02-09-1971	14:02:30	34.41 N	118.40 W	D	69	8.0	4.3
02-09-1971	14:02:31	34.41 N	118.40 W	D	69	8.0	4.7
02-09-1971	14:02:44	34.41 N	118.40 W	D	69	8.0	5.8
02-09-1971	14:03:25	34.41 N	118.40 W	D	69	8.0	4.4
02-09-1971	14:03:46	34.41 N	118.40 W	D	69	8.0	4.1
02-09-1971	14:04:07	34.41 N	118.40 W	D	69	8.0	4.1
02-09-1971	14:04:34	34.41 N	118.40 W	C	69	8.0	4.2
02-09-1971	14:04:39	34.41 N	118.40 W	D	69	8.0	4.1
02-09-1971	14:04:44	34.41 N	118.40 W	D	69	8.0	4.1
02-09-1971	14:04:46	34.41 N	118.40 W	D	69	8.0	4.2
02-09-1971	14:05:41	34.41 N	118.40 W	D	69	8.0	4.1
02-09-1971	14:05:50	34.41 N	118.40 W	D	69	8.0	4.1
02-09-1971	14:07:10	34.41 N	118.40 W	D	69	8.0	4.0
02-09-1971	14:07:30	34.41 N	118.40 W	D	69	8.0	4.0
02-09-1971	14:07:45	34.41 N	118.40 W	D	69	8.0	4.5
02-09-1971	14:08:04	34.41 N	118.40 W	D	69	8.0	4.0
02-09-1971	14:08:07	34.41 N	118.40 W	D	69	8.0	4.2
02-09-1971	14:08:38	34.41 N	118.40 W	D	69	8.0	4.5

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Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(CAL TECH DATA 1932-2003)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
02-09-1971	14:08:53	34.41 N	118.40 W	D	69	8.0	4.6
02-09-1971	14:10:21	34.36 N	118.31 W	B	62	5.0	4.7
02-09-1971	14:10:28	34.41 N	118.40 W	D	69	8.0	5.3
02-09-1971	14:16:12	34.34 N	118.33 W	C	60	11.1	4.1
02-09-1971	14:19:50	34.36 N	118.41 W	B	64	11.8	4.0
02-09-1971	14:34:36	34.34 N	118.64 W	C	72	-2.0	4.9
02-09-1971	14:39:17	34.39 N	118.36 W	C	66	-1.6	4.0
02-09-1971	14:40:17	34.43 N	118.40 W	C	72	-2.0	4.1
02-09-1971	14:43:46	34.31 N	118.45 W	B	60	6.2	5.2
02-09-1971	15:58:20	34.33 N	118.33 W	B	60	14.2	4.8
02-09-1971	16:19:26	34.46 N	118.43 W	B	75	-1.0	4.2
02-10-1971	03:12:12	34.37 N	118.30 W	B	63	.8	4.0
02-10-1971	05:06:36	34.41 N	118.33 W	A	68	4.7	4.3
02-10-1971	05:18:07	34.43 N	118.41 W	A	71	5.8	4.5
02-10-1971	11:31:34	34.38 N	118.46 W	A	68	6.0	4.2
02-10-1971	13:49:53	34.40 N	118.42 W	A	69	9.7	4.3
02-10-1971	14:35:26	34.36 N	118.49 W	A	67	4.4	4.2
02-10-1971	17:38:55	34.40 N	118.37 W	A	67	6.2	4.2
02-10-1971	18:54:41	34.45 N	118.44 W	A	74	8.1	4.2
02-21-1971	05:50:52	34.40 N	118.44 W	A	69	6.9	4.7
02-21-1971	07:15:11	34.39 N	118.43 W	A	68	7.2	4.5
03-07-1971	01:33:40	34.35 N	118.46 W	A	65	3.3	4.5
03-25-1971	22:54:09	34.36 N	118.47 W	A	66	4.6	4.2
03-30-1971	08:54:43	34.30 N	118.46 W	A	59	2.6	4.1
03-31-1971	14:52:22	34.29 N	118.51 W	A	61	2.1	4.6
04-01-1971	15:03:03	34.43 N	118.41 W	A	72	8.0	4.1
04-02-1971	05:40:25	34.28 N	118.53 W	A	61	3.0	4.0
04-15-1971	11:14:32	34.26 N	118.58 W	B	62	4.2	4.2
04-25-1971	14:48:06	34.37 N	118.31 W	B	63	-2.0	4.0
06-21-1971	16:01:08	34.27 N	118.53 W	B	60	4.1	4.0
06-22-1971	10:41:19	33.75 N	117.48 W	B	66	8.0	4.2
02-21-1973	14:45:57	34.06 N	119.04 W	B	83	8.0	5.3
03-09-1974	00:54:31	34.40 N	118.47 W	C	70	24.4	4.7
08-14-1974	14:45:55	34.43 N	118.37 W	A	71	8.2	4.2
01-01-1976	17:20:12	33.97 N	117.89 W	A	33	6.2	4.2

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DATE	TIME	LATITUDE		LONGITUDE		Q	DIST	DEPTH	MAGNITUDE
04-08-1976	15:21:38	34.35	N	118.66	W	A	73	14.5	4.6
08-12-1977	02:19:26	34.38	N	118.46	W	B	68	9.5	4.5
09-24-1977	21:28:24	34.46	N	118.41	W	C	75	5.0	4.2
05-23-1978	09:16:50	33.91	N	119.17	W	C	91	6.0	4.0
01-01-1979	23:14:38	33.94	N	118.68	W	B	48	11.3	5.2
10-17-1979	20:52:37	33.93	N	118.67	W	C	47	5.5	4.2
10-19-1979	12:22:37	34.21	N	117.53	W	B	75	4.9	4.1
09-04-1981	15:50:50	33.65	N	119.09	W	C	86	6.0	5.5
10-23-1981	17:28:17	33.64	N	119.01	W	C	78	6.0	4.6
10-23-1981	19:15:52	33.62	N	119.02	W	A	80	14.8	4.6
04-13-1982	11:02:12	34.06	N	118.97	W	A	77	12.1	4.0
05-25-1982	13:44:30	33.55	N	118.21	W	A	29	12.6	4.3
01-08-1983	07:19:30	34.13	N	117.45	W	A	77	7.8	4.1
02-22-1983	02:18:30	33.03	N	117.94	W	D	89	10.0	4.3
02-27-1984	10:18:15	33.47	N	118.06	W	C	40	6.0	4.0
10-26-1984	17:20:43	34.02	N	118.99	W	A	77	13.3	4.6
10-02-1985	23:44:12	34.02	N	117.25	W	A	90	15.2	4.8
07-13-1986	13:47:08	32.97	N	117.87	W	C	98	6.0	5.4
07-13-1986	14:01:33	32.99	N	117.84	W	C	96	6.0	4.3
07-30-1986	22:51:13	32.99	N	117.80	W	C	98	6.0	4.0
07-31-1986	01:06:19	32.97	N	117.83	W	C	99	6.0	4.1
09-30-1986	09:52:11	32.99	N	117.80	W	C	98	6.0	4.1
02-21-1987	23:15:29	34.13	N	117.45	W	A	77	8.5	4.0
10-01-1987	14:42:20	34.06	N	118.08	W	A	30	9.5	5.9
10-01-1987	14:45:41	34.05	N	118.10	W	A	28	13.6	4.7
10-01-1987	14:48:03	34.08	N	118.09	W	A	31	11.7	4.1
10-01-1987	14:49:05	34.06	N	118.10	W	A	29	11.7	4.7
10-01-1987	15:12:31	34.05	N	118.09	W	A	28	10.8	4.7
10-01-1987	15:59:53	34.05	N	118.09	W	A	28	10.4	4.0
10-04-1987	10:59:38	34.07	N	118.10	W	A	30	8.3	5.3
10-24-1987	23:58:33	33.68	N	119.06	W	A	82	12.2	4.1
02-11-1988	15:25:55	34.08	N	118.05	W	A	32	12.5	4.7
06-26-1988	15:04:58	34.14	N	117.71	W	A	57	7.9	4.7
11-20-1988	05:39:28	33.51	N	118.07	W	C	35	6.0	4.9
12-03-1988	11:38:26	34.15	N	118.13	W	A	38	14.3	5.0

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DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
01-19-1989	06:53:28	33.92 N	118.63 W	A	42	11.9	5.0
02-18-1989	07:17:04	34.01 N	117.74 W	A	47	3.3	4.1
04-07-1989	20:07:30	33.62 N	117.90 W	A	34	12.9	4.7
06-12-1989	16:57:18	34.03 N	118.18 W	A	24	15.6	4.6
06-12-1989	17:22:25	34.02 N	118.18 W	A	23	15.5	4.4
12-28-1989	09:41:08	34.19 N	117.39 W	A	85	14.6	4.3
02-28-1990	23:43:36	34.14 N	117.70 W	A	59	4.5	5.4
03-01-1990	00:34:57	34.13 N	117.70 W	A	57	4.4	4.0
03-01-1990	03:23:03	34.15 N	117.72 W	A	58	11.4	4.7
03-02-1990	17:26:25	34.15 N	117.69 W	A	59	5.6	4.7
04-04-1990	08:54:39	32.97 N	117.81 W	C	100	6.0	4.3
04-17-1990	22:32:27	34.11 N	117.72 W	A	54	3.6	4.8
06-28-1991	14:43:54	34.27 N	117.99 W	A	54	9.1	5.8
06-28-1991	17:00:55	34.25 N	117.99 W	A	52	9.5	4.3
07-05-1991	17:41:57	34.50 N	118.56 W	A	83	10.9	4.1
01-17-1994	12:30:55	34.21 N	118.54 W	A	55	18.4	6.7
01-17-1994	12:30:55	34.22 N	118.54 W	A	55	17.4	6.6
01-17-1994	12:31:58	34.27 N	118.49 W	C	59	6.0	5.9
01-17-1994	12:34:18	34.31 N	118.47 W	C	61	6.0	4.4
01-17-1994	12:39:39	34.26 N	118.54 W	C	60	6.0	4.9
01-17-1994	12:40:09	34.32 N	118.51 W	C	64	6.0	4.8
01-17-1994	12:40:36	34.34 N	118.61 W	C	71	6.0	5.2
01-17-1994	12:54:33	34.31 N	118.46 W	C	60	6.0	4.0
01-17-1994	12:55:46	34.28 N	118.58 W	C	63	6.0	4.1
01-17-1994	13:06:28	34.25 N	118.55 W	C	59	6.0	4.6
01-17-1994	13:26:45	34.32 N	118.46 W	C	61	6.0	4.7
01-17-1994	13:28:13	34.27 N	118.58 W	C	62	6.0	4.0
01-17-1994	13:56:02	34.29 N	118.62 W	C	67	6.0	4.4
01-17-1994	14:14:30	34.33 N	118.44 W	C	62	6.0	4.5
01-17-1994	15:07:03	34.30 N	118.47 W	A	61	2.6	4.2
01-17-1994	15:07:35	34.31 N	118.47 W	A	61	1.6	4.1
01-17-1994	15:54:10	34.38 N	118.63 W	A	75	13.0	4.8
01-17-1994	17:56:08	34.23 N	118.57 W	A	58	19.2	4.6
01-17-1994	19:35:34	34.31 N	118.46 W	A	61	2.3	4.0
01-17-1994	19:43:53	34.37 N	118.64 W	A	74	13.9	4.1

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DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
01-17-1994	20:46:02	34.30 N	118.57 W	C	65	6.0	4.9
01-17-1994	22:31:53	34.34 N	118.44 W	C	63	6.0	4.1
01-17-1994	23:33:30	34.33 N	118.70 W	A	74	9.8	5.6
01-17-1994	23:49:25	34.34 N	118.67 W	A	74	8.4	4.0
01-18-1994	00:39:35	34.38 N	118.56 W	A	72	7.2	4.4
01-18-1994	00:40:04	34.39 N	118.54 W	A	72	.0	4.2
01-18-1994	00:43:08	34.38 N	118.70 W	A	78	11.3	5.2
01-18-1994	04:01:26	34.36 N	118.62 W	A	73	.9	4.3
01-18-1994	07:23:56	34.33 N	118.62 W	A	70	14.8	4.0
01-18-1994	11:35:09	34.22 N	118.61 W	A	59	12.1	4.2
01-18-1994	13:24:44	34.32 N	118.56 W	A	66	1.7	4.3
01-18-1994	15:23:46	34.38 N	118.56 W	A	72	7.7	4.8
01-19-1994	04:40:48	34.36 N	118.57 W	A	71	2.6	4.3
01-19-1994	04:43:14	34.37 N	118.71 W	C	78	6.0	4.0
01-19-1994	09:13:10	34.30 N	118.74 W	A	75	13.0	4.1
01-19-1994	14:09:14	34.22 N	118.51 W	A	54	17.5	4.5
01-19-1994	21:09:28	34.38 N	118.71 W	A	79	14.4	5.1
01-19-1994	21:11:44	34.38 N	118.62 W	A	74	11.4	5.1
01-21-1994	18:39:15	34.30 N	118.47 W	A	60	10.6	4.5
01-21-1994	18:39:47	34.30 N	118.48 W	A	60	11.9	4.0
01-21-1994	18:42:28	34.31 N	118.47 W	A	61	7.9	4.2
01-21-1994	18:52:44	34.30 N	118.45 W	A	60	7.6	4.3
01-21-1994	18:53:44	34.30 N	118.46 W	A	59	7.7	4.3
01-23-1994	08:55:08	34.30 N	118.43 W	A	59	6.0	4.1
01-24-1994	04:15:18	34.35 N	118.55 W	A	68	6.5	4.6
01-24-1994	05:50:24	34.36 N	118.63 W	A	73	12.1	4.3
01-24-1994	05:54:21	34.36 N	118.63 W	A	74	10.9	4.2
01-27-1994	17:19:58	34.27 N	118.56 W	A	62	14.9	4.6
01-28-1994	20:09:53	34.38 N	118.49 W	A	69	.7	4.2
01-29-1994	11:20:35	34.31 N	118.58 W	A	66	1.1	5.1
01-29-1994	12:16:56	34.28 N	118.61 W	A	65	2.7	4.3
02-03-1994	16:23:35	34.30 N	118.44 W	A	59	9.0	4.0
02-05-1994	08:51:29	34.37 N	118.65 W	A	75	15.4	4.0
02-06-1994	13:19:27	34.29 N	118.48 W	A	60	9.3	4.1
02-25-1994	12:59:12	34.36 N	118.48 W	A	66	1.2	4.0

NOTE: Q IS A FACTOR RELATING THE QUALITY OF EPICENTRAL DETERMINATION

A = +- 1 km horizontal distance; +- 2 km depth
B = +- 2 km horizontal distance; +- 5 km depth
C = +- 5 km horizontal distance; no depth restriction
D = >+- 5 km horizontal distance

Event qualities are highly suspect prior to 1990. Many of these event qualities are based on incomplete information according to Caltech.

Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(CAL TECH DATA 1932-2003)

DATE	TIME	LATITUDE		LONGITUDE		Q	DIST	DEPTH	MAGNITUDE
03-20-1994	21:20:12	34.23	N	118.47	W	A	54	13.1	5.2
05-25-1994	12:56:57	34.31	N	118.39	W	A	59	7.0	4.4
06-15-1994	05:59:48	34.31	N	118.40	W	A	59	7.4	4.1
12-06-1994	03:48:34	34.29	N	118.39	W	A	57	9.0	4.5
02-19-1995	21:24:18	34.05	N	118.92	W	A	72	15.6	4.3
06-21-1995	21:17:36	32.98	N	117.82	W	C	98	6.0	4.3
06-26-1995	08:40:28	34.39	N	118.67	W	A	78	13.3	5.0
03-20-1996	07:37:59	34.36	N	118.61	W	A	73	13.0	4.1
05-01-1996	19:49:56	34.35	N	118.70	W	A	77	14.4	4.1
04-26-1997	10:37:30	34.37	N	118.67	W	A	76	16.5	5.1
04-26-1997	10:40:29	34.37	N	118.67	W	A	77	14.6	4.0
04-27-1997	11:09:28	34.38	N	118.65	W	A	76	15.2	4.8
06-28-1997	21:45:25	34.17	N	117.34	W	A	88	10.0	4.2
01-05-1998	18:14:06	33.95	N	117.71	W	A	47	11.5	4.3
03-11-1998	12:18:51	34.02	N	117.23	W	A	92	14.9	4.5
08-20-1998	23:49:58	34.37	N	117.65	W	A	80	9.0	4.4
07-22-1999	09:57:24	34.40	N	118.61	W	A	76	11.6	4.0
02-21-2000	13:49:43	34.05	N	117.26	W	A	90	15.0	4.5
03-07-2000	00:20:28	33.81	N	117.72	W	A	44	11.3	4.0
01-14-2001	02:26:14	34.28	N	118.40	W	A	56	8.8	4.3
01-14-2001	02:50:53	34.29	N	118.40	W	A	57	8.4	4.0
09-09-2001	23:59:18	34.06	N	118.39	W	A	33	7.9	4.2
10-28-2001	16:27:45	33.92	N	118.27	W	A	14	21.1	4.0
12-14-2001	12:01:35	33.95	N	117.75	W	A	44	13.8	4.0
01-29-2002	05:53:28	34.36	N	118.66	W	A	75	14.1	4.2
09-03-2002	07:08:51	33.92	N	117.78	W	A	40	12.9	4.8

NOTE: Q IS A FACTOR RELATING THE QUALITY OF EPICENTRAL DETERMINATION

A = +- 1 km horizontal distance; +- 2 km depth
B = +- 2 km horizontal distance; +- 5 km depth
C = +- 5 km horizontal distance; no depth restriction
D = >+- 5 km horizontal distance

Event qualities are highly suspect prior to 1990. Many of these event qualities are based on incomplete information according to Caltech.

Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(CAL TECH DATA 1932-2003)

S E A R C H   O F   E A R T H Q U A K E   D A T A   F I L E 1

SITE: Long Beach Memorial Medical Center

COORDINATES OF SITE	33.8113 N 118.1889 W
DISTANCE PER DEGREE	110.9 KM-N 92.6 KM-W
MAGNITUDE LIMITS	4.0 - 8.5
TEMPORAL LIMITS	1932 - 2003
SEARCH RADIUS (KM)	100
NUMBER OF YEARS OF DATA	72.00
NUMBER OF EARTHQUAKES IN FILE	4187
NUMBER OF EARTHQUAKES IN AREA	411

MACTEC Engineering and Consulting

Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(RICHTER DATA 1906-1931)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
05-15-1910	15:47:00	33.70 N	117.40 W	D	74	.0	6.0
07-23-1923	07:30:26	34.00 N	117.25 W	D	89	.0	6.3

S E A R C H   O F   E A R T H Q U A K E   D A T A   F I L E   2

SITE: Long Beach Memorial Medical Center

COORDINATES OF SITE	33.8113 N 118.1889 W
DISTANCE PER DEGREE	110.9 KM-N 92.6 KM-W
MAGNITUDE LIMITS	6.0 - 8.5
TEMPORAL LIMITS	1906 - 1931
SEARCH RADIUS (KM)	100
NUMBER OF YEARS OF DATA	26.00
NUMBER OF EARTHQUAKES IN FILE	35
NUMBER OF EARTHQUAKES IN AREA	2

MACTEC Engineering and Consulting

Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(NOAA/CDMG DATA 1812-1905)

DATE	TIME	LATITUDE	LONGITUDE	Q	DIST	DEPTH	MAGNITUDE
02-09-1890	04:06:00	34.00 N	117.50 W	D	67	.0	7.0

S E A R C H   O F   E A R T H Q U A K E   D A T A   F I L E   3

SITE: Long Beach Memorial Medical Center

COORDINATES OF SITE	33.8113 N 118.1889 W
DISTANCE PER DEGREE	110.9 KM-N 92.6 KM-W
MAGNITUDE LIMITS	7.0 - 8.5
TEMPORAL LIMITS	1812 - 1905
SEARCH RADIUS (KM)	100
NUMBER OF YEARS OF DATA	94.00
NUMBER OF EARTHQUAKES IN FILE	9
NUMBER OF EARTHQUAKES IN AREA	1

MACTEC Engineering and Consulting

Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(NOAA/CDMG DATA 1812-1905)

S U M M A R Y   O F   E A R T H Q U A K E   S E A R C H

* * *

NUMBER OF HISTORIC EARTHQUAKES WITHIN 100 KM RADIUS OF SITE

MAGNITUDE RANGE	NUMBER
4.0 - 4.5	274
4.5 - 5.0	94
5.0 - 5.5	31
5.5 - 6.0	8
6.0 - 6.5	3
6.5 - 7.0	3
7.0 - 7.5	1
7.5 - 8.0	0
8.0 - 8.5	0

* * *

MACTEC Engineering and Consulting

Table 3
List Of Historic Earthquakes Of Magnitude 4.0 Or Greater Within 100 Km Of The Site
(NOAA/CDMG DATA 1812-1905)

C O M P U T A T I O N   O F   R E C U R R E N C E   C U R V E

L O G   N = A - B M

* * *

BIN	MAGNITUDE	RANGE	NO/YR (N)
1	4.00	4.00 - 8.50	5.70
2	4.50	4.50 - 8.50	1.89
3	5.00	5.00 - 8.50	.588
4	5.50	5.50 - 8.50	.157
5	6.00	6.00 - 8.50	.460E-01
6	6.50	6.50 - 8.50	.358E-01
7	7.00	7.00 - 8.50	.521E-02 NU
8	7.50	7.50 - 8.50	.000
9	8.00	8.00 - 8.50	.000

A = 1.186   B = .5783   (NORMALIZED)
A = 4.463   B = .9385   SIGMA = .141

* * *

Table 4: Pseudospectral Velocity in Inches/Second

	2% damping		5% damping		10% damping	
Period in Seconds	DBE 10% in 50 years	MCE 10% in 100 years	DBE 10% in 50 years	MCE 10% in 100 years	DBE 10% in 50 years	MCE 10% in 100 years
0.01	0.23	0.30	0.23	0.30	0.23	0.30
0.05	1.62	2.14	1.62	2.14	1.62	2.14
0.10	5.19	6.78	4.34	5.67	3.69	4.83
0.20	14.56	18.80	11.25	14.53	8.74	11.29
0.30	21.33	27.83	16.91	22.07	13.57	17.71
0.40	25.84	34.31	21.05	27.94	17.42	23.13
0.50	29.64	40.05	24.14	32.62	19.98	27.01
0.75	36.98	50.75	30.12	41.34	24.94	34.22
1.00	41.49	57.39	33.79	46.74	27.97	38.69
2.00	46.44	63.75	39.41	54.09	34.08	46.79
3.00	42.92	59.34	36.42	50.35	31.50	43.55
4.00	37.95	53.20	32.20	45.14	27.85	39.04

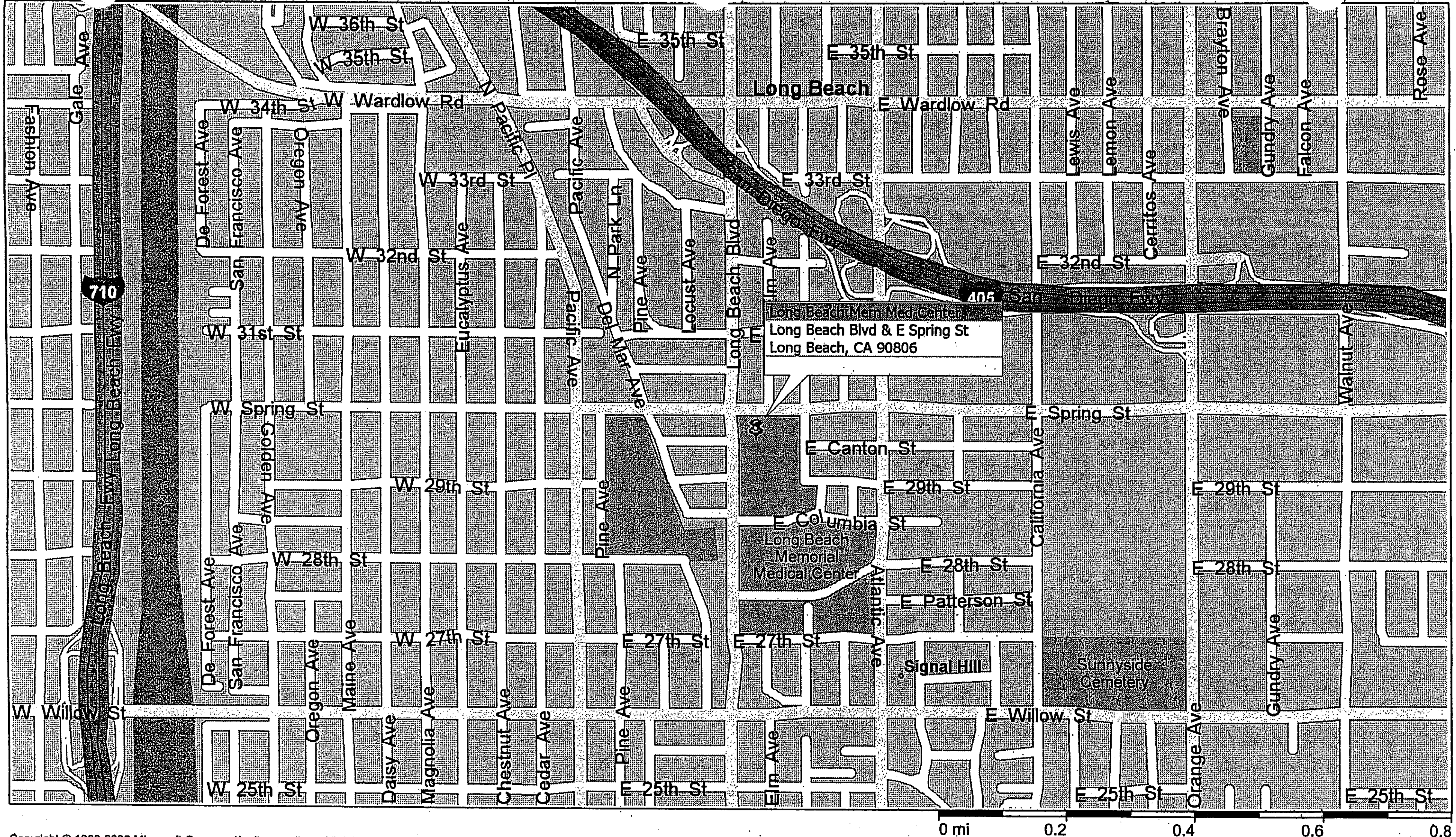
By JAA 10/8/04
Chkd KR 10/19/04

Table 5: Pseudospectral Acceleration in g

	2% damping		5% damping		10% damping	
Period in Seconds	DBE 10% in 50 years	MCE 10% in 100 years	DBE 10% in 50 years	MCE 10% in 100 years	DBE 10% in 50 years	MCE 10% in 100 years
0.01	0.37	0.49	0.37	0.49	0.37	0.49
0.05	0.53	0.70	0.53	0.70	0.53	0.70
0.10	0.84	1.10	0.71	0.92	0.60	0.79
0.20	1.18	1.53	0.91	1.18	0.71	0.92
0.30	1.16	1.51	0.92	1.20	0.74	0.96
0.40	1.05	1.39	0.86	1.14	0.71	0.94
0.50	0.96	1.30	0.79	1.06	0.65	0.88
0.75	0.80	1.10	0.65	0.90	0.54	0.74
1.00	0.67	0.93	0.55	0.76	0.45	0.63
2.00	0.38	0.52	0.32	0.44	0.28	0.38
3.00	0.23	0.32	0.20	0.27	0.17	0.24
4.00	0.15	0.22	0.13	0.18	0.11	0.16

By JAA 10/8/04
Chkd KR 10/19/04

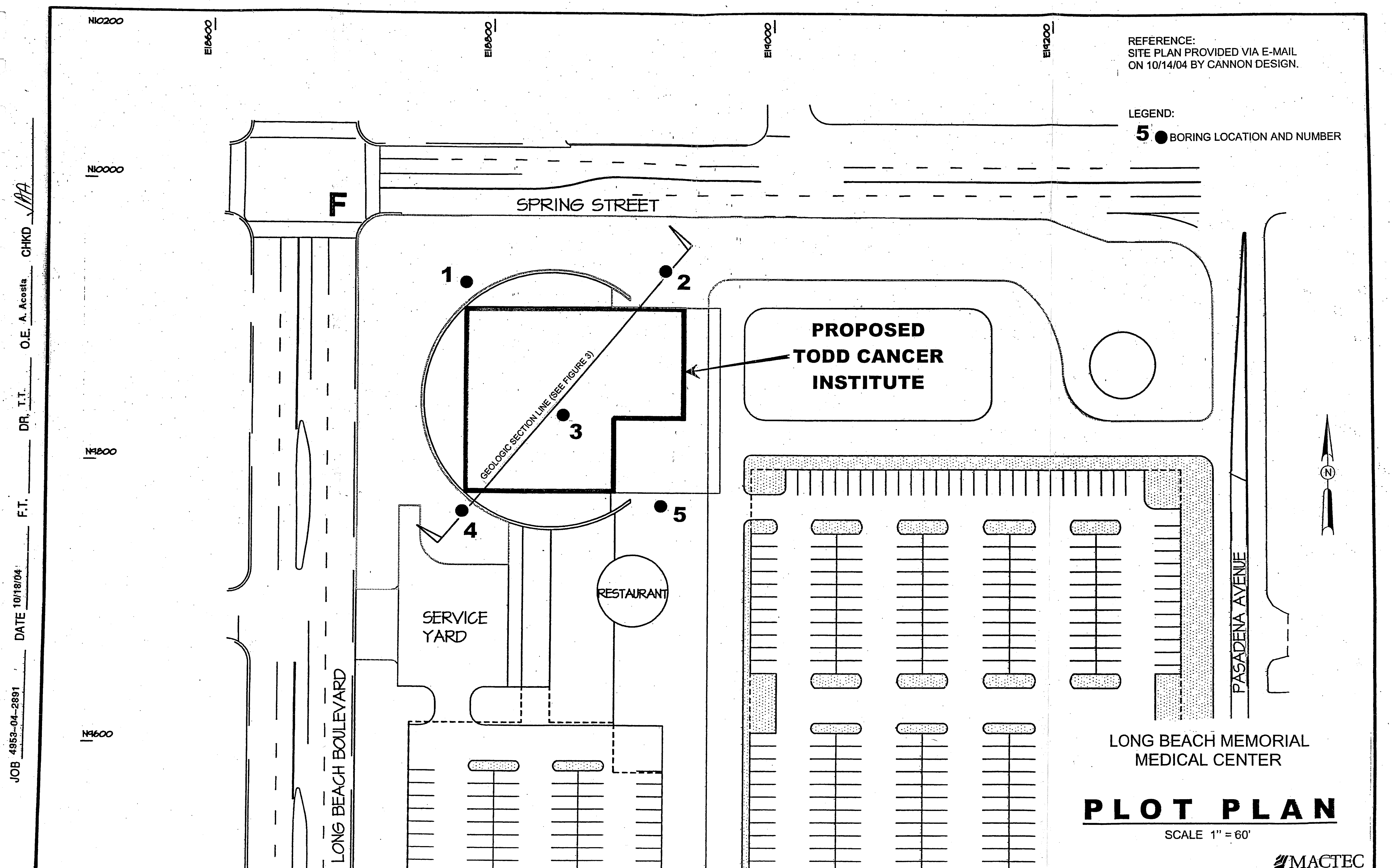
FIGURES



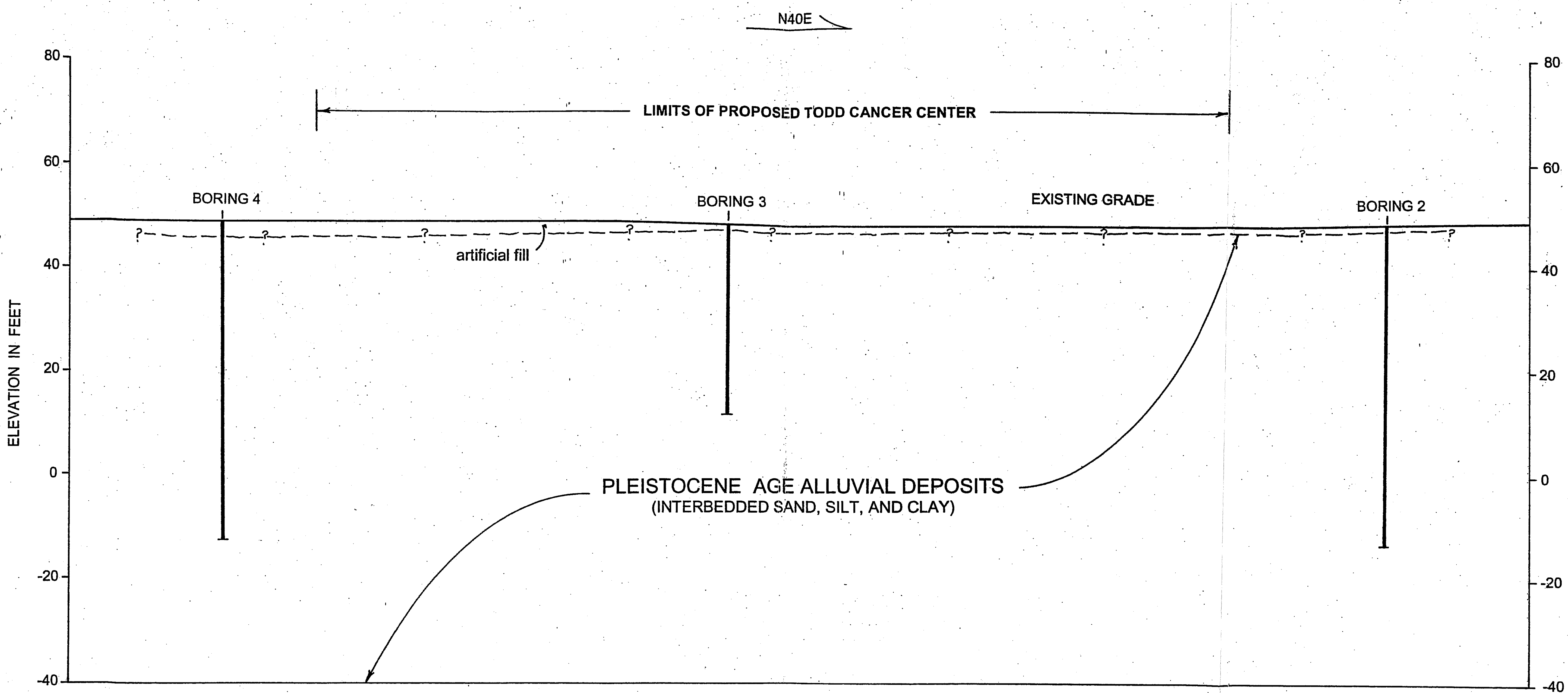
VICINITY MAP

MACTEC

FIGURE 1


N10200
E18600
E18800
E19000
E19200
N10000
N9800
N9600
REFERENCE:
SITE PLAN PROVIDED VIA E-MAIL
ON 10/14/04 BY CANNON DESIGN.
LEGEND:
5 BORING LOCATION AND NUMBER
SPRING STREET
F
PROPOSED
TODD CANCER
INSTITUTE
GEOLOGIC SECTION LINE (SEE FIGURE 3)
1
2
3
4
5
SERVICE
YARD
RESTAURANT
LONG BEACH BOULEVARD
PASADENA AVENUE
N
LONG BEACH MEMORIAL
MEDICAL CENTER
PLOT PLAN
SCALE 1" = 60'
MACTEC
JOB 4953-04-2891
DATE 10/18/04
F.T.
DR. T.T.
O.E. A. Acosta
CHKD

JOB 4953-04-2891 DATE 10/21/04 F.T. DR. T.T. O.E. K.Rice CHKD



NOTES:

1. SECTION BASED ON SOIL CONDITIONS AT BORING LOCATIONS. SOIL CONDITIONS BETWEEN BORINGS HAVE BEEN INTERPOLATED AND LOCALIZED VARIATIONS FROM CONDITIONS ENCOUNTERED MAY OCCUR. SECTION IS INTENDED FOR DESCRIPTIVE PURPOSES ONLY.

2. SEE FIGURE 2 FOR LOCATION OF SECTION.

GEOLOGIC SECTION

SCALE 1" = 20'

MACTEC

EXPLANATION:

Qya2 YOUNGER ALLUVIAL DEPOSITS

Qoa OLDER ALLUVIAL DEPOSITS

GEOLOGIC CONTACT

FAULT

COORDINATES:
Latitude N33.8113
Longitude W118.1889

REFERENCES:

CALIFORNIA DIVISION OF MINES AND GEOLOGY, 1998, "SEISMIC HAZARD EVALUATION OF THE LONG BEACH 7.5-MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA," OFR 98-19.

POLAND, J. F. AND PIPER, A. M., 1956, "GROUND-WATER GEOLOGY OF THE COASTAL ZONE LONG BEACH-SANTA ANA AREA, CALIFORNIA," USGS WATER SUPPLY PAPER 1109.

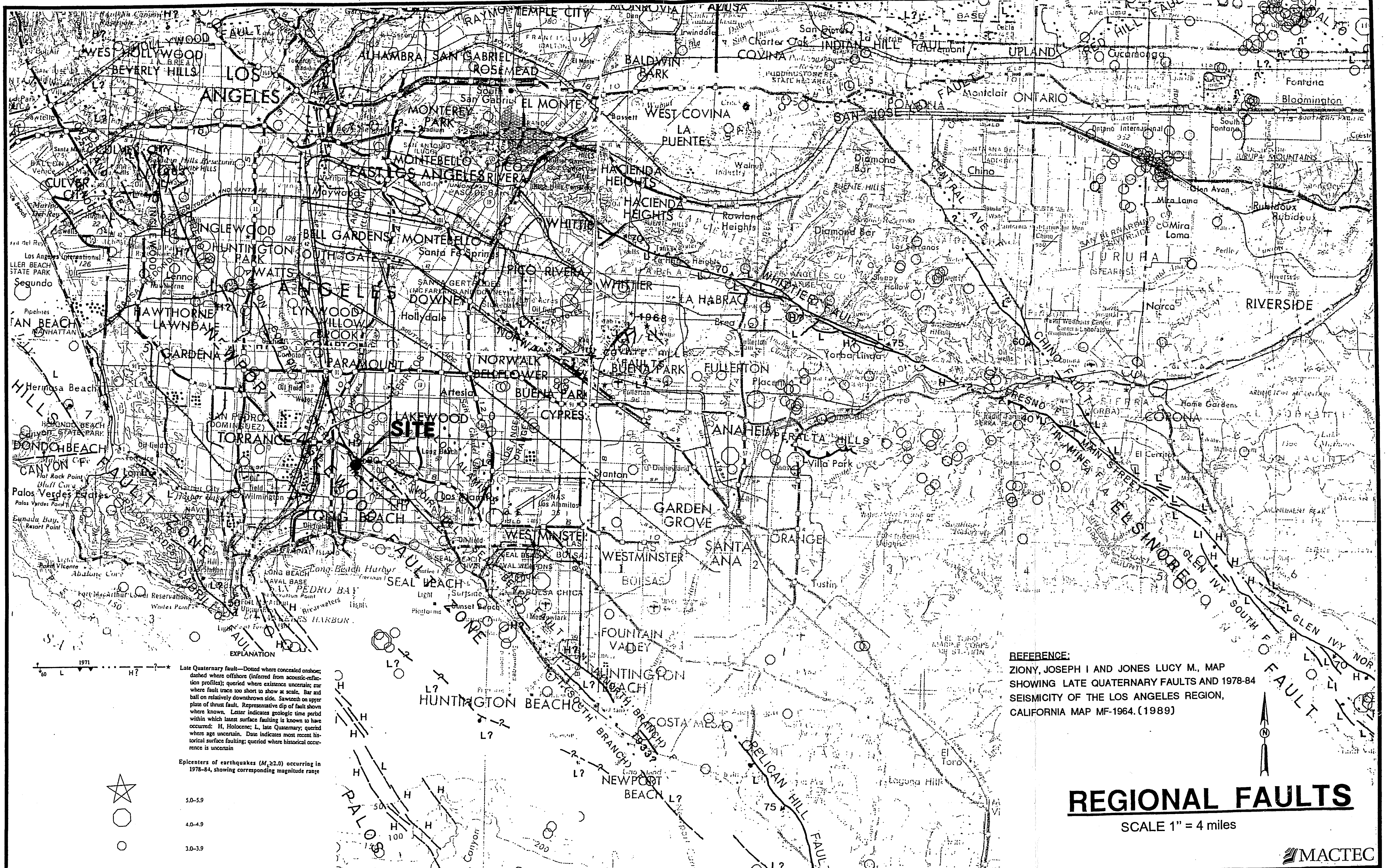
USGS, 1964, "7.5-MINUTE LONG BEACH QUADRANGLE MAP," PHOTOREVISED 1981.

LOCAL GEOLOGY

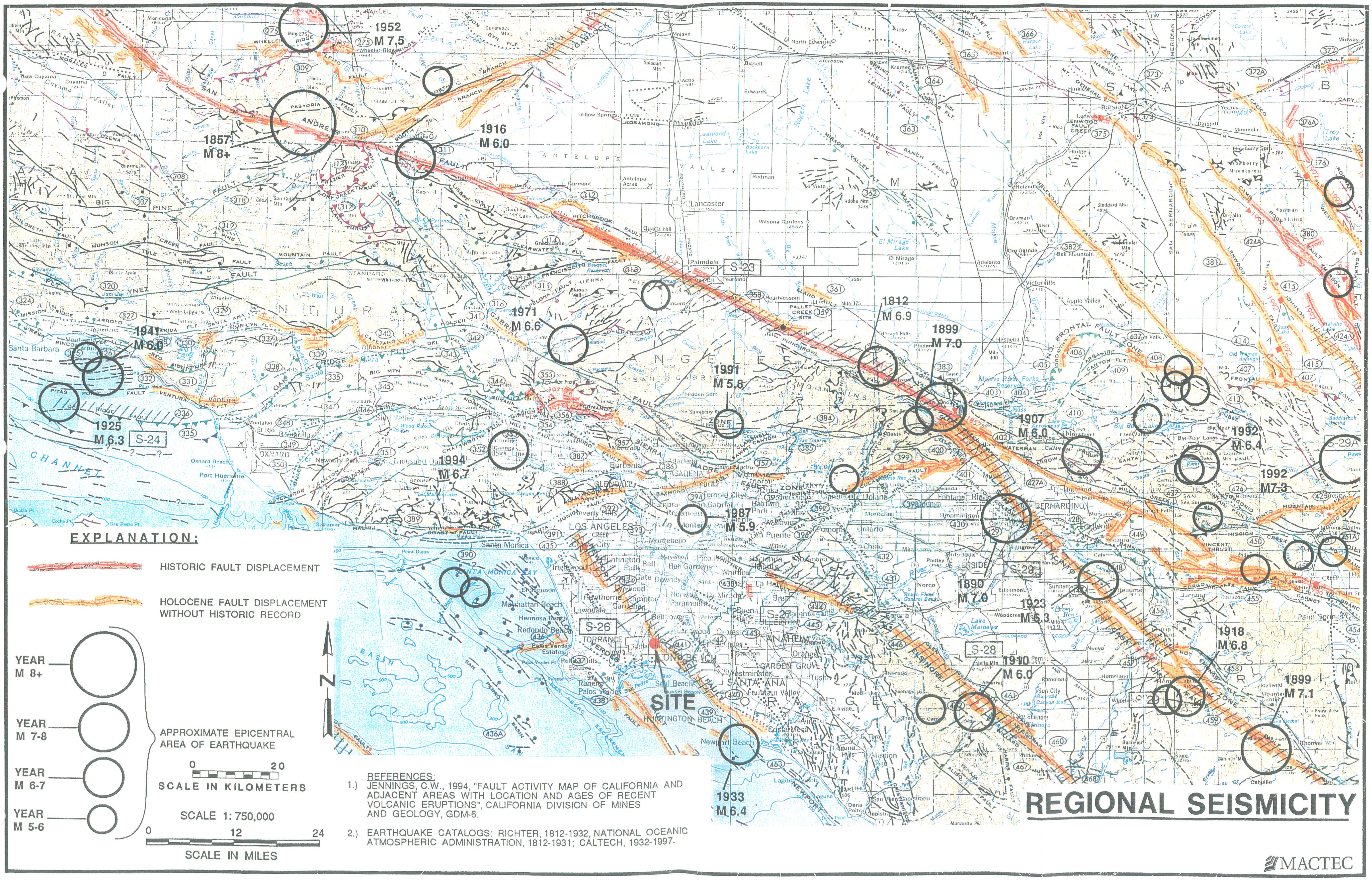
SCALE 1" = 2000' (0.61 km)

MACTEC

FIGURE 4



LOS ANGELES
WEST HOLLYWOOD
BEVERLY HILLS
ALHAMBRA
SAN GABRIEL
ROSEMEAD
TEMPLE CITY
MONROVIA
AZUSA
COVINA
BALDWIN PARK
WEST COVINA
LA PUENTE
EL MONTE
MONTEREY PARK
MONTEBELLO
EAST LOS ANGELES
PICO RIVERA
HACIENDA HEIGHTS
WHITTIER
UPLAND
ONTARIO
Montclair
POMONA
Chino
Fontana
Bloomington
RIVERSIDE
CORONA
Norco
CULVER CITY
INGLEWOOD
HUNTINGTON PARK
BELL GARDENS
SOUTH GATE
WATTS
DOWNEY
HAWTHORNE
LAWNDALE
LYNWOOD
GARDENA
PARAMOUNT
NORWALK
BELLFLOWER
LA HABRA
BUENA PARK
FULLERTON
Artesia
LAKEWOOD
SITE
CYPRESS
ANAHEIM
TORRANCE
LONG BEACH
SEAL BEACH
WESTMINSTER
GARDEN GROVE
SANTA ANA
ORANGE
Tustin
FOUNTAIN VALLEY
HUNTINGTON BEACH
COSTA MESA
NEWPORT BEACH
Hermosa Beach
Palos Verdes Estates
SAN PEDRO BAY
WHITTIER FAULT
CHINO FAULT
ELSINORE FAULT
NEWPORT-INGLEWOOD FAULT ZONE
PALOS VERDES HILLS FAULT
PELICAN HILL FAULT
INDIAN HILL FAULT
RED HILL FAULT
SAN JOSE FAULT
GLEN IVY SOUTH FAULT
GLEN IVY NORTH FAULT
EXPLANATION
1971
60
L
H?
Late Quaternary fault—Dotted where concealed onshore; dashed where offshore (inferred from acoustic-reflection profiles); queried where existence uncertain; bar where fault trace too short to show at scale. Bar and ball on relatively downthrown side. Sawteeth on upper plate of thrust fault. Representative dip of fault shown where known. Letter indicates geologic time period within which latest surface faulting is known to have occurred: H, Holocene; L, late Quaternary; queried where age uncertain. Date indicates most recent historical surface faulting; queried where historical occurrence is uncertain
Epicenters of earthquakes ($M_L \geq 2.0$) occurring in 1978–84, showing corresponding magnitude range
5.0–5.9
4.0–4.9
3.0–3.9
REFERENCE:
ZIONY, JOSEPH I AND JONES LUCY M., MAP SHOWING LATE QUATERNARY FAULTS AND 1978-84 SEISMICITY OF THE LOS ANGELES REGION, CALIFORNIA MAP MF-1964. (1989)
N
REGIONAL FAULTS
SCALE 1" = 4 miles
MACTEC

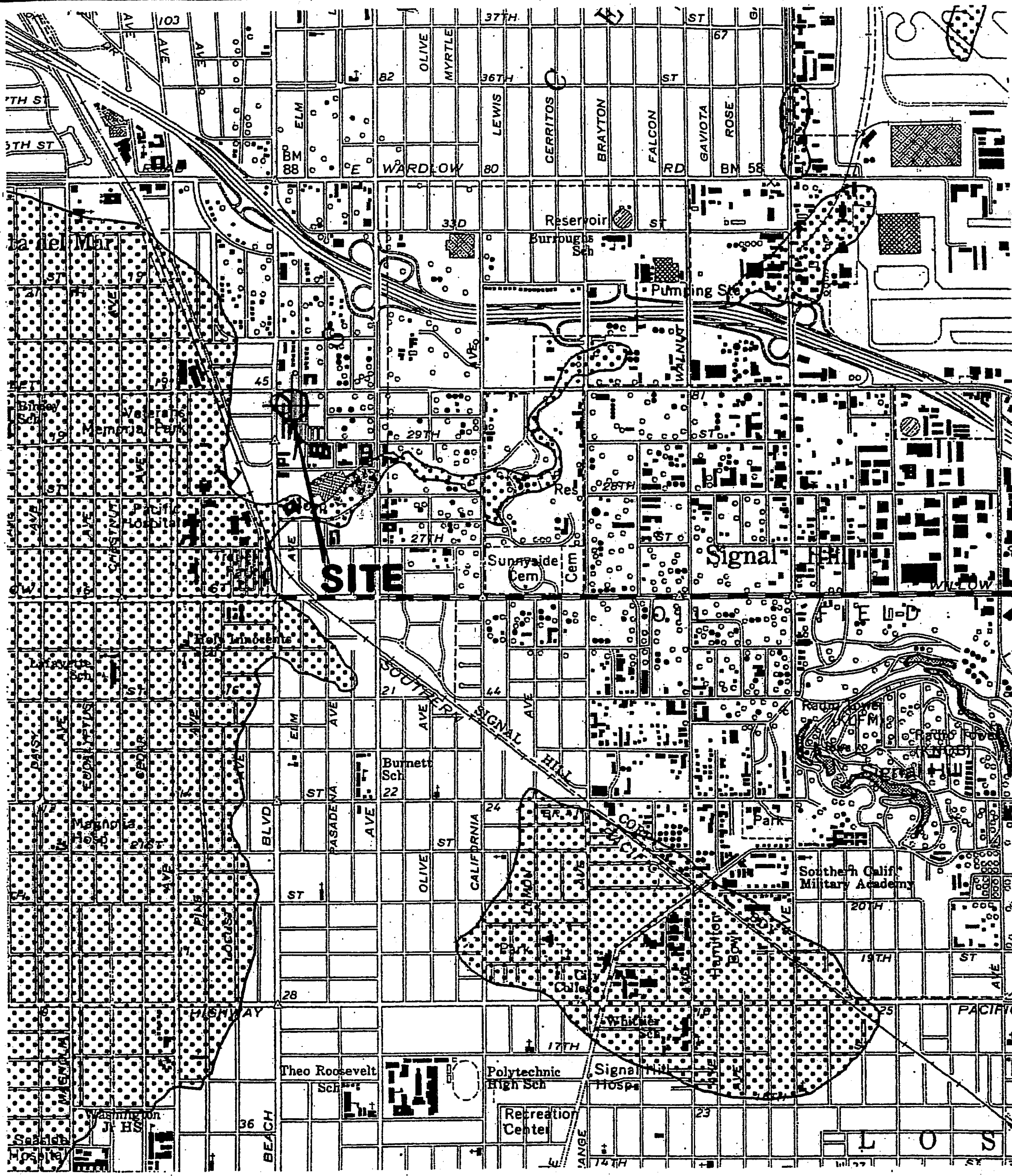


REGIONAL SEISMICITY

MACTEC

FIGURE 6

JOB 4953-04-2091 DATE 11/04 F.T. DR. O.E. CHKD



MAP EXPLANATION: Zones of Required Investigation

Liquefaction

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake–Induced Landslides

Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

REFERENCE:
California Division of Mines and Geology, 1999, "State of California Seismic Hazard Zones, Long Beach Quadrangle, Official Map."

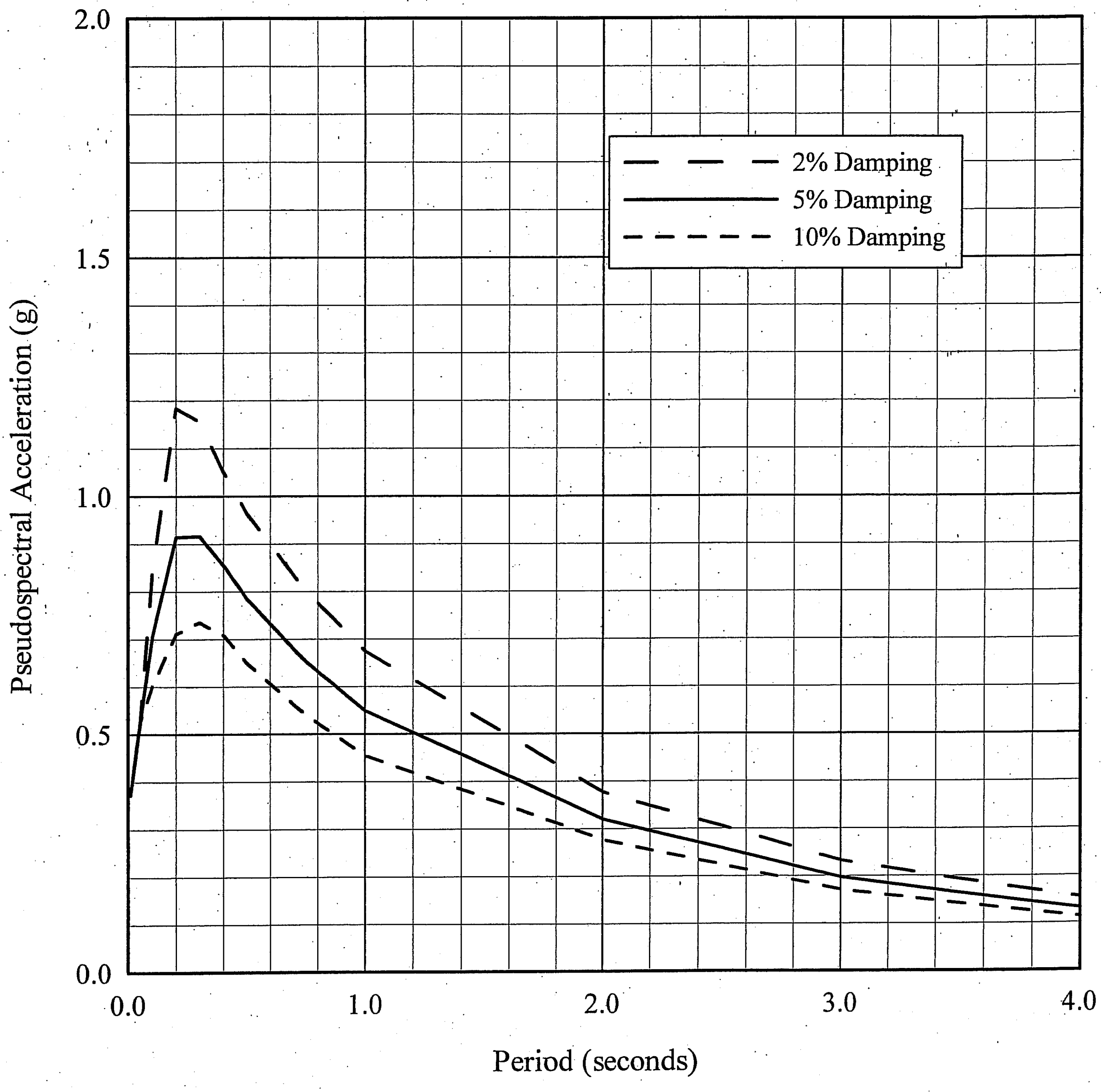
N

SEISMIC HAZARDS ZONE MAP

SCALE 1" = 2000' (0.61 km)

MACTEC

FIGURE 7



RESPONSE SPECTRA
Pseudospectral Acceleration

Design Basis Earthquake (DBE)
10% Probability of Being Exceeded in 50 Years

MACTEC

JOB: 4953-04-2891 DATE: October 8, 2004 DR.: JAA O.E.: JAA CHKD: KR 10/19/04

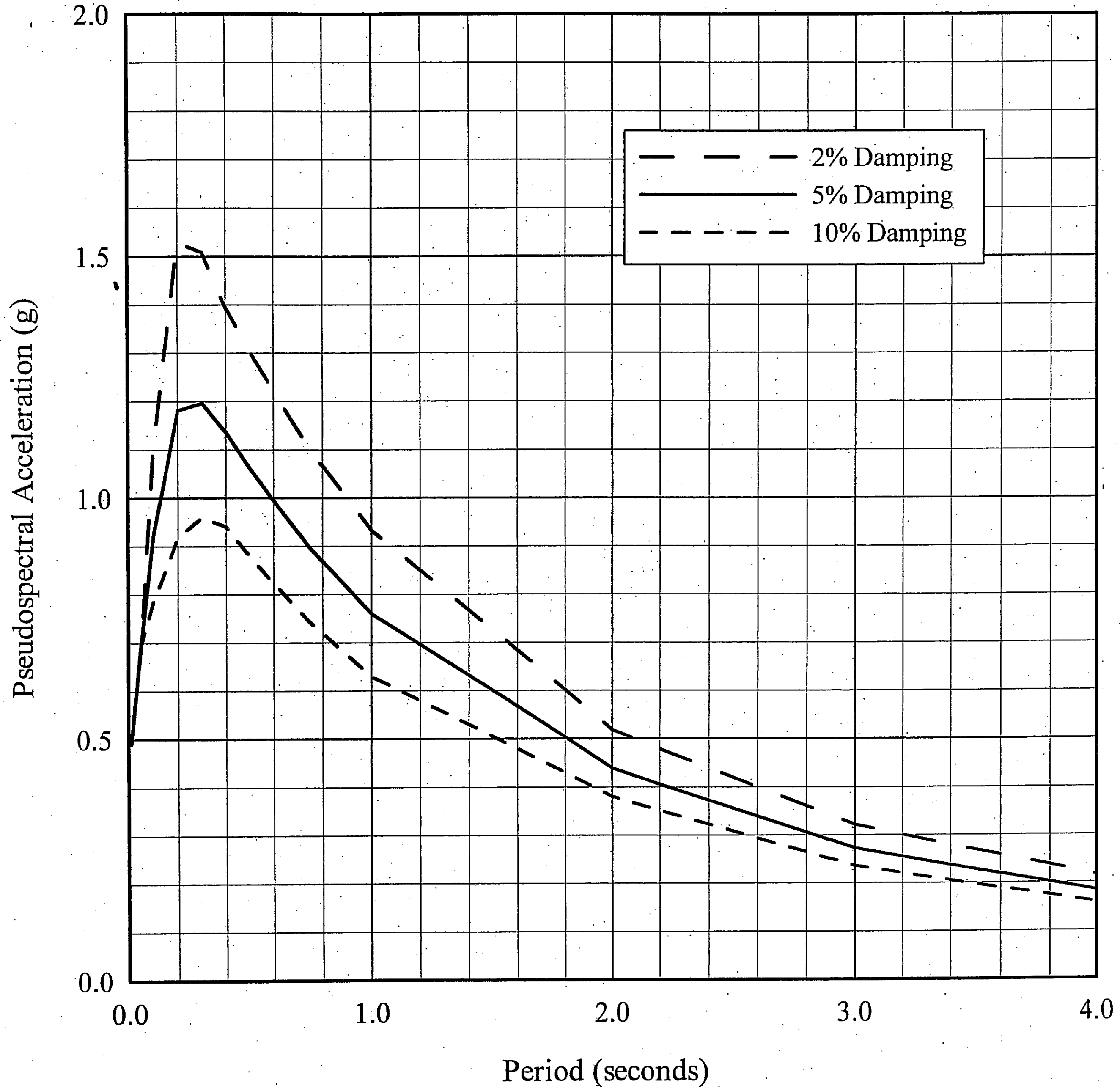
CHKD: [illegible] 10/19/04

O.E.: JAA

DR.: JAA

DATE: October 8, 2004

JB: 4953-04-2891



RESPONSE SPECTRA
Pseudospectral Acceleration

Upper Bound Earthquake (UBE)
10% Probability of Being Exceeded in 100 Years

MACTEC

APPENDIX

EXPLORATIONS AND LABORATORY TESTS

APPENDIX

EXPLORATIONS AND LABORATORY TESTS

EXPLORATIONS

The soil conditions beneath the site were explored by drilling five borings. The locations of our borings are depicted in Figure 2. The current borings were drilled to depths between 35 and 61.5 feet below the existing grade using 8-inch-diameter hollow-stem-auger-type drilling equipment. Caving and raveling of the boring walls did not occur; casing or drilling mud was not used to extend the borings to the depths drilled.

The soils encountered were logged by our field technician and undisturbed and bulk samples were obtained for laboratory inspection and testing. The logs of the current borings are presented in Figures A-1.1 through A-1.5. The depths at which the undisturbed samples were obtained are indicated on the left of the boring logs. The number of blows required to drive the Crandall sampler 12 inches using a 140 hammer falling 30 inches is indicated on the logs. The soils are classified in accordance with the Unified Soil Classification System described in Figure A-2.

LABORATORY TESTS

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to determine their engineering properties.

The field moisture content and dry density of the soils encountered were determined by performing tests on the undisturbed samples. The results of the tests are depicted on the left of the boring logs.

Direct shear tests were performed on selected undisturbed samples to determine the strength of the soils. The tests were performed at field moisture content and after soaking to near-saturated moisture content and at various surcharge pressures. The yield-point values determined from the direct shear tests are presented in Figure A-3.1, Direct Shear Test Data.

Confined consolidation tests were performed on three undisturbed samples to determine the compressibility of the soils. Water was added to one of the samples during the tests to illustrate the effect of moisture on the compressibility. The results of the tests are presented in Figures A-4.1 through A-4.2, Consolidation Test Data.

The optimum moisture content and maximum dry density of the upper soils were determined by performing a compaction test on a sample obtained from Boring B-4. The test was performed in accordance with the ASTM Designation D1557-91 method of compaction. The results of the tests are presented in Figure A-5, Compaction Data.

The Expansion Index of the soils was determined by testing one sample in accordance with the Uniform Building Code Standard No. 29-2 method. The results of the test are presented in Figure A-6, Expansion Index Test Data.

To provide information for paving design, a stabilometer test ("R" value test) was performed on a sample of the upper soils. The test was performed for us by LaBelle-Marvin Professional Pavement Engineering. The results of the test are presented in Figures A-7.1 and A-7.2.

Soil corrosivity studies were performed on samples of the on-site soils. The results of the study and recommendations for mitigating procedures are presented in Figures A-8.1 through A-8.6.

BORING 1

DATE DRILLED: September 28, 2004
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION: 48.0 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		DESCRIPTION
								FILL - SILTY SAND - very loose, very moist, medium brown, fine to medium, some clay balls, planter soil
							SM	SILTY SAND - very loose, very moist, reddish brown, fine
45					10			
								clayey lens
	5	31					ML	CLAYEY SILT - hard, moist, dark to medium brown, fine sand
40			15.1	118	56			
								slightly sandier
	10	19					SM	SILTY SAND - very dense, very moist, medium brown, fine, some clay
35			18.1	117	80/11"		ML	SANDY SILT - hard, moist, grayish green, fine sand sandy lens
	15	37					SM	SILTY SAND - very dense, moist, greenish gray, fine to medium
								some clean sand seams
30			11.1	105	48			
	20	83/9"					SP	POORLY-GRADED SAND - very dense, moist, grayish green, fine
25			7.8	97	81			
	25	91					ML	SANDY SILT - hard, moist, grayish green, fine sand, silty sand seems
20			23.8	101	58			
	30	81					SC	CLAYEY SAND - very hard, moist, greenish gray, fine to coarse
15			9.5	128	86			
	35	80						End of boring at 36½'
10								
	40							

NOTES: 1. Water not encountered. 2. Borehole backfilled with soil cuttings and tamped.

* Number of blows required to drive the Crandall sampler 12 inches using a 140 pound hammer falling 30 inches

** Elevations are based on Site Plan provided by Cannon Design

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Field Tech: AR
Prepared By: LT
Checked By: JMcK

BORING 2

DATE DRILLED: September 27, 2004
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION: 48.5 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	USCS	DESCRIPTION
								3" Thick Asphalt Concrete
								FILL - 12" Thick, disturbed natural, concrete fragments
							SM	SILTY SAND - medium dense, moist, medium brown, fine
45			4.6	106	31			
	5	29						slightly porous
								some silty and clayey seams
40			5.2	109	55			
	10	51						
							ML	SANDY SILT - stiff, moist, light brownish gray, fine sand
35					27			
							SM	SILTY SAND - medium dense, moist, light brownish gray, fine
	15	21						
30			5.0	101	50			17.9% Passing Sieve No. 200
								less silt
	20	81/9"					SP	POORLY-GRADED SAND - very hard, moist, light gray, fine to medium, silty seams
25			3.7	92	89/10"			trace of shell fragments
							CL-ML	SILTY CLAY - hard, moist, medium gray, very fine sand
	25	55						
20			12.1	121	90/10"		SM-SC	SILTY SAND - very dense, moist, greenish gray, fine to medium
	30	89/11"						
15			7.6	122	100/9"			23.3% Passing Sieve No. 200
	35	83/11"						slightly coarse, some silt balls
							ML	CLAYEY SILT - hard, moist, brownish gray, very fine sand
10			24.3	102	87/11"			
	40							

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Field Tech: AR
Prepared By: LT
Checked By: VMcK

(CONTINUED ON FOLLOWING FIGURE)

BORING 2 (Continued)

DATE DRILLED: September 27, 2004
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION: 48.5 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		DESCRIPTION
		57				☒	ML	SANDY SILT - hard, moist, brownish gray, very fine sand
5			12.4	116	100/9"	▩		
	45	79				☒		some clayey seams
0			11.9	122	89/9"	▩	SM	SILTY SAND - very dense, wet, fine to medium
								wet
	50	40				☒	ML	SANDY SILT - hard, wet, medium brown, very fine sand, some clay
								clayey seams
-5			29.1	95	87/9"	▩		
	55	87				☒	SP -SM	sand seems, very dense, wet SILTY SAND - very dense, wet, olive brown, very fine
-10			20.7	102	100	▩		
	60	82				☒		
								End of Boring at 61½'
-15	65							
-20	70							
-25	75							
-30	80							

NOTES: 1. Water encountered at 55'. 2. Borehole backfilled with soil cuttings and tamped.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Field Tech: AR
Prepared By: LT
Checked By:

BORING 3

DATE DRILLED: September 27, 2004
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION: 48.0 **

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	USCS	DESCRIPTION
							3" Thick Asphalt Concrete
							12" Thick Crush Rock and Asphalt Concrete
						ML	SANDY SILT - hard, moist, medium brown, fine to medium sand, some clay
45			13.5	125	82	ML -CL	CLAYEY SILT - hard, moist, medium brown
	5	43					
			11.3	121	63		
40							less clay
	10	22				SM	SILTY SAND - dense, moist, light to medium brown, fine to medium, silt seams
						ML	CLAYEY SILT - hard, moist, medium gray, fine sand
35			33.5	90	63		
	15						grayish brown, very fine sand seams
		53				SM	SILTY SAND - very dense, moist, light brownish gray, fine to medium, some clean sand seams
			6.6	118	83/11"		
30							
	20	97				SP	POORLY-GRADED SAND - very dense, moist, light brownish yellow, very fine to fine
25			2.7	95	88/9"		
	25	50					some shell fragments
						SM	SILTY SAND - dense, moist, medium brown, fine to medium
							brown and gray
20			10.1	123	75/9"		
	30	85					some clay
15			12.2	123	95/9"		
	35	58				SP -SM	POORLY-GRADED SAND - very dense, moist, medium brown, fine to medium
							End boring at 36½'
10							
	40						

NOTES: 1. Water not encountered. 2. Borehole backfilled soil cuttings and tamped.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL_CRANDAL V1 42891.GPJ LAW_CRAN.GDT 10/22/04

Field Tech: AR
Prepared By: LT
Checked By: JMcK

Long Beach Memorial Hospital
Long Beach, California

MACTEC

LOG OF BORING
Project: 4953-04-2891 Figure: A-1.3

BORING 4

DATE DRILLED: September 28, 2004
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION: 49.0 **

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	USCS	DESCRIPTION
								FILL - SANDY SILT with CLAY - stiff, moist, dark brown, fine to medium sand, some 4" brick concrete fragments
							SM	SILTY SAND - medium dense, moist, reddish brown, fine
45			14.0	118	25			
	5	32						
							ML	SANDY SILT - very stiff, moist, dark brown, very fine sand, some clay
			15.7	119	57			
40								
	10	25						sandy seams
			36.4	85	65			some cemented silt fragments
35								
	15	81/11"						
			26.1	98	85/11"		SM	SILTY SAND - very dense, moist, light brown, fine to medium
30								
	20	72						
							ML	POORLY-GRADED SAND - very dense, moist, light brown, fine to medium
			7.0	94	100/9"			
25								
	25	90/11"						
							SM -ML	SILTY SAND - very dense, moist, brown and greenish gray, fine
			15.0	117	84			47.1% Passing Sieve No. 200
20								
	30	67						
							SC	CLAYEY SAND - very dense, moist, brown and greenish gray, fine
			15.2	115	93			
15								
	35						SM	SILTY SAND - very dense, moist, medium to light brown, fine
		90/11"						
			18.1	108	80/9"			
10								
	40							

Field Tech: AR
Prepared By: LT
Checked By: JMcK

(CONTINUED ON FOLLOWING FIGURE)

BORING 4 (Continued)

DATE DRILLED: September 28, 2004
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION: 49.0 **

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	USCS	DESCRIPTION
		55						
							ML	SANDY SILT - hard, moist, brown and gray, very fine sand
5			23.5	99	64			
	45	62					CL	SILTY CLAY - hard, moist, bluish gray, very fine sand
					64		SM	SILTY SAND - dense, moist, light brown, fine
0	50	77					CL	SILTY CLAY - hard, moist, light brown, fine sand
			31.0	93	100			
-5							SP	POORLY-GRADED SAND - very dense, very moist, medium grayish brown, fine to medium
	55	87					ML	CLAYEY SILT - hard, moist, brown and gray, very fine sand
			27.5	94	93			
-10	60	94/11"						
-15	65							
-20	70							
-25	75							
-30	80							

End of Boring at 61½'

NOTES: 1. Water encountered at 53'. 2. Borehole backfilled with soil cuttings and tamped.

Field Tech: AR
Prepared By: LT
Checked By: [illegible]

SOIL CRANDAL. /1 42891.GPJ LAW_CRAN.GDT 10/22/04

BORING 5

DATE DRILLED: September 27, 2004
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.): 8
ELEVATION: 48.5 **

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	USCS	DESCRIPTION
	0					SM	3" Thick Asphalt Concrete SILTY SAND - medium dense, moist, medium brown, fine, silty seams
						ML	CLAYEY SILT - very stiff, moist, brownish orange, very fine sand, some silty clay
45			12.7	117	45		
	5	43					
						SM	SILTY SAND - medium dense, moist, medium brown, fine
40			11.9	116	23	ML	SANDY SILT - stiff, moist, light to medium gray, fine sand
	10	26					
							soft
35			19.2	101	19		
							sandy seams
	15	77/11"				SM	SILTY SAND - dense, moist, light to medium gray, fine
			7.9	102	75/9"		
30							
	20	88/11"				SP	POORLY-GRADED SAND - dense, moist, light brown, fine to medium
						SM	SILTY SAND - very dense, moist, light brown and gray, fine to medium, some silty seams
25			16.2	107	87/10"		
	25	75					some clay
			13.9	120	92/11"		
20							
	30	74					
15			7.7	113	95/11"		
	35	84					End of Boring at 35'
10							
	40						

NOTES: Water not encountered. 2. Borehole backfilled with soil cuttings and tamped.

Field Tech: AR
Prepared By: LT
Checked By:

Long Beach Memorial Hospital
Long Beach, California

MACTEC

LOG OF BORING
Project: 4953-04-2891
Figure: A-1.5

\JOR DIVISIONS			GROUP SYMBOLS	TYPICAL \IES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel - sand mixtures, little or no fines.
			GP	Poorly graded gravels or grave - sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel - sand - silt mixtures.
			GC	Clayey gravels, gravel - sand - clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 Sieve Size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand - silt mixtures
			SC	Clayey sands, sand - clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts and with slight plasticity.
			CL	Inorganic lays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

SILT OR CLAY	SAND			GRAVEL		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

No.200 No.40 No.10 No.4 3/4" 3" 12"

U.S. STANDARD SIEVE SIZE

Reference: The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953 (Revised April, 1960)

Undisturbed Sample	Auger Cut
Split Spoon Sample	Bulk Sample
Rock Core	Crandall Sampler
Dilatometer	Pressure Meter
Packer	No Recovery
Water Table at time of drilling	Water Table after 24 hours

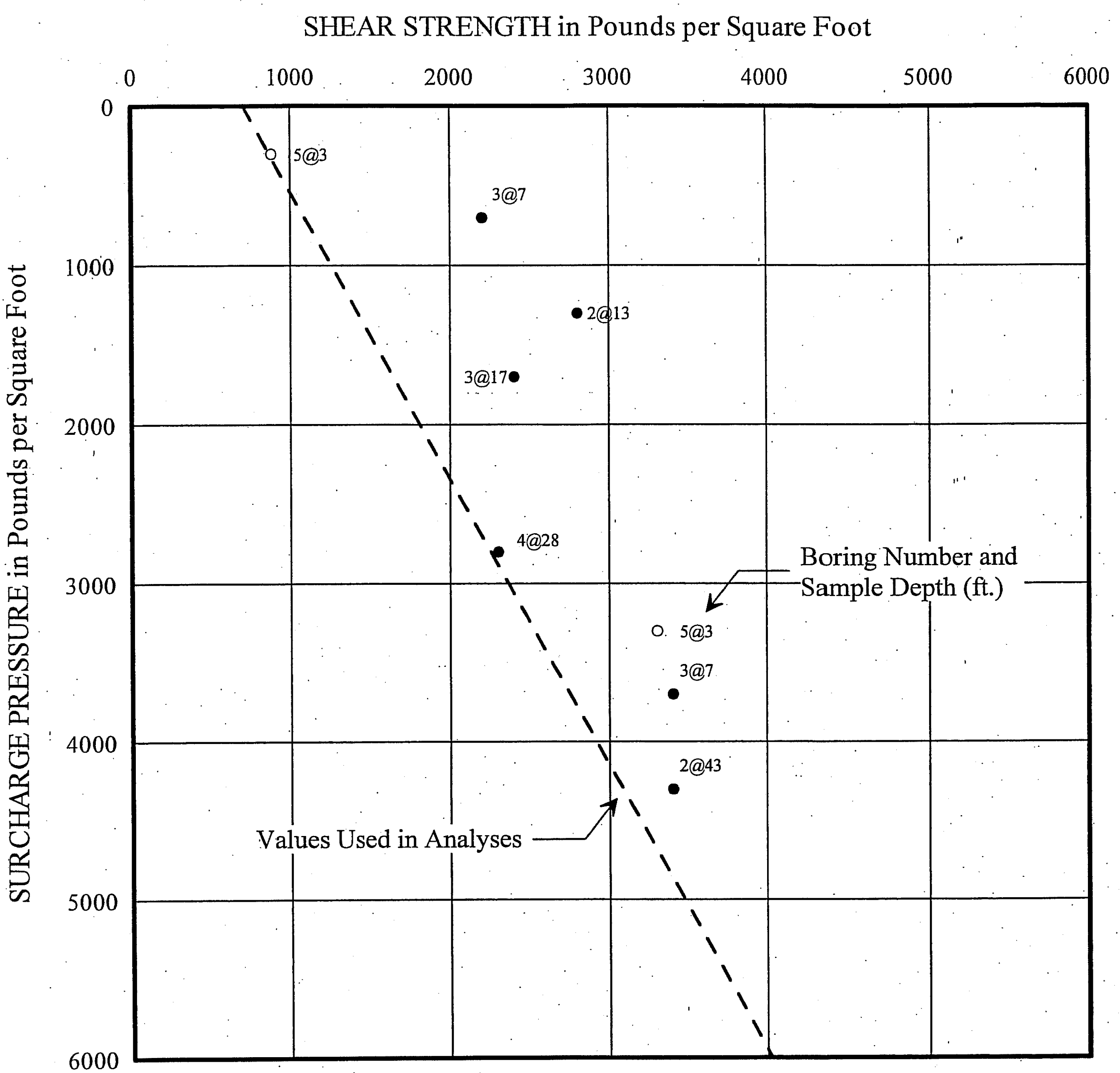
Correlation of Penetration Resistance with Relative Density and Consistency

SAND & GRAVEL		SILT & CLAY	
No. of Blows	Relative Density	No. of Blows	Consistency
0 - 4	Very Loose	0 - 1	Very Soft
5 - 10	Loose	2 - 4	Soft
11 - 20	Firm	5 - 8	Firm
21 - 30	Very Firm	9 - 15	Stiff
31 - 50	Dense	16 - 30	Very Stiff
Over 50	Very Dense	Over 31	Hard

KEY TO SYMBOLS AND DESCRIPTIONS

MACTEC

FIGURE A-2



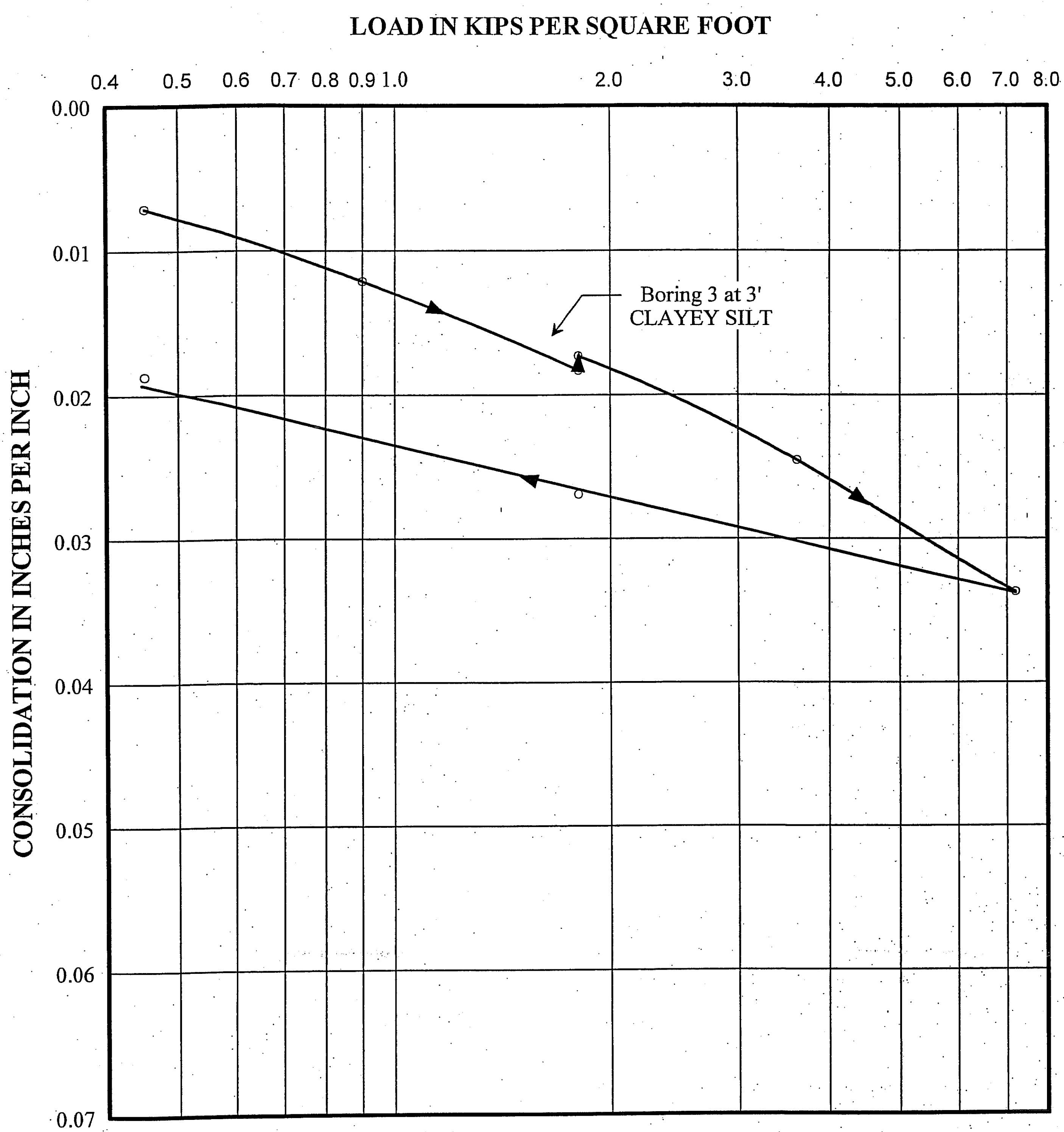
KEY:
- ○ Samples tested after soaking to a moisture content near saturation
- ● Samples tested at field mositure content

DIRECT SHEAR TEST DATA

OB 4953-04-2891 DATE October 13, 2004 F.T. DR. LT O.E. JAA CHKD JAA

MACTEC

FIGURE A - 3

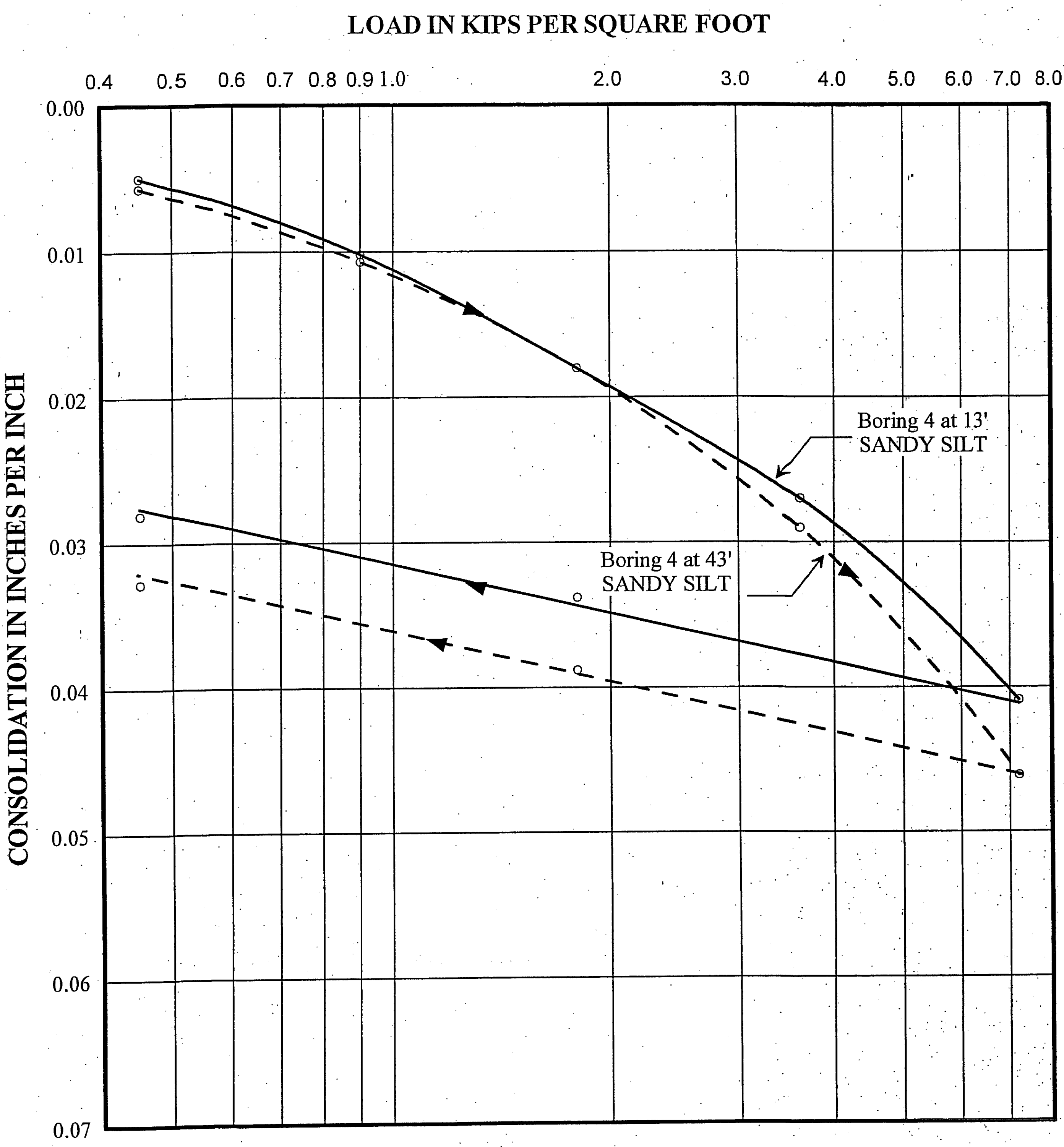


NOTE: Water added to sample after consolidation under a load of 1.8 kips per square foot.

CONSOLIDATION TEST DATA

MACTEC

FIGURE A - 4.1



CONSOLIDATION TEST DATA

MACTEC

BORING NUMBER AND SAMPLE DEPTH:	4 at 3' to 6'
SOIL TYPE:	SILTY SAND
MAXIMUM DRY DENSITY: (lbs./cu.ft.)	131
OPTIMUM MOISTURE CONTENT: (%)	8.2

TEST METHOD: ASTM Designation D1557

COMPACTION TEST DATA

MACTEC

BORING NUMBER AND SAMPLE DEPTH:	5 at 3'
SOIL TYPE:	CLAYEY SILT
CONFINING PRESSURE: (lbs./sq. ft.)	144
INITIAL MOISTURE CONTENT: (% dry wt.)	11.7
FINAL MOISTURE CONTENT: (% dry wt.)	22.4
DRY DENSITY: (lbs/cu.ft.)	107.1
EXPANSION INDEX:	68

EXPANSION INDEX TEST DATA

MACTEC

R - VALUE DATA SHEET

P.N.4953-04-2891.02
L.B. Mem. Hosp.

PROJECT NUMBER 31588 BORING NUMBER: B-1 @ 2'-7'

SAMPLE DESCRIPTION: Brown Silty Sand

Item	SPECIMEN a	SPECIMEN b	SPECIMEN c
Mold Number	16	17	18
Water added, grams	86	79	74
Initial Test Water, %	10.1	9.5	9.1
Compact Gage Pressure,psi	160	350	350
Exudation Pressure, psi	219	331	456
Height Sample, Inches	2.62	2.50	2.48
Gross Weight Mold, grams	3270	3244	3213
Tare Weight Mold, grams	2099	2101	2079
Sample Wet Weight, grams	1171	1143	1134
Expansion, Inches x 10exp-4	0	7	23
Stability 2,000 lbs (160psi)	29 / 58	25 / 50	23 / 43
Turns Displacement	5.38	4.93	4.30
R-Value Uncorrected	45	53	61
R-Value Corrected	48	53	61
Dry Density, pcf	123.0	126.5	127.0
	DESIGN CALCULATION DATA		
Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.53	0.48	0.40
G. E. by Expansion	0.00	0.23	0.77

Equilibrium R-Value 51 by EXUDATION

Examined & Checked: 10 /14/ 04

REGISTERED PROFESSIONAL ENGINEER
STEVEN R. MARVIN
No. 30659
Exp. 03/31/06
CIVIL
STATE OF CALIFORNIA

Steven R. Marvin, RCE 30659

REMARKS:

Gf = 1.25
0.0 % retained on the
3/4" sieve.

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle • Marvin

FIGURE 7.1

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 31588
P.N. 4953-04-2891.02

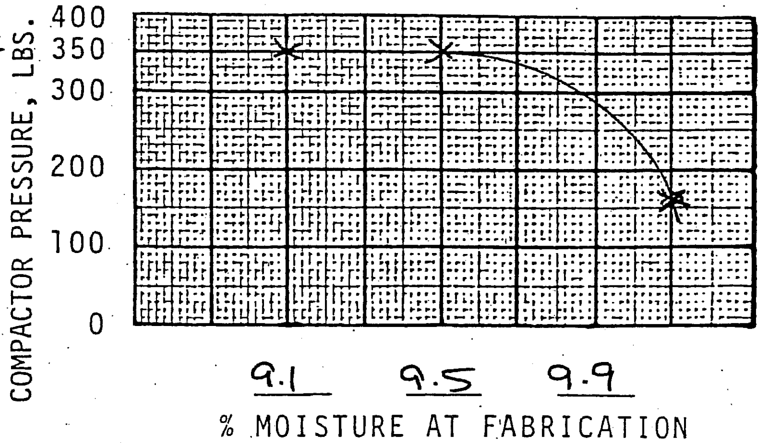
BORING NO. B-1 @ 2'-7'

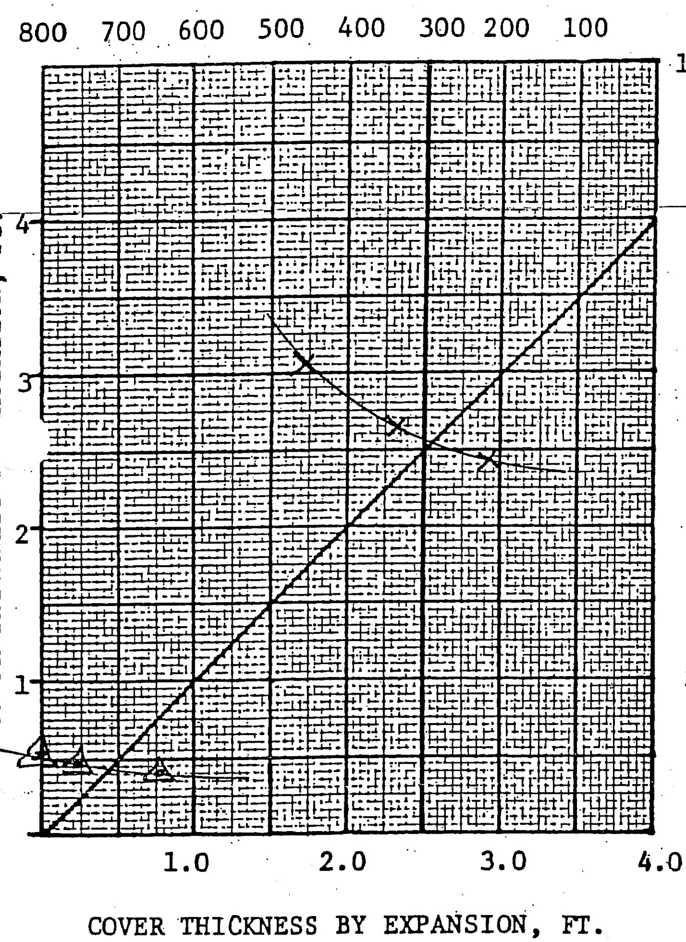
DATE 10-14-04

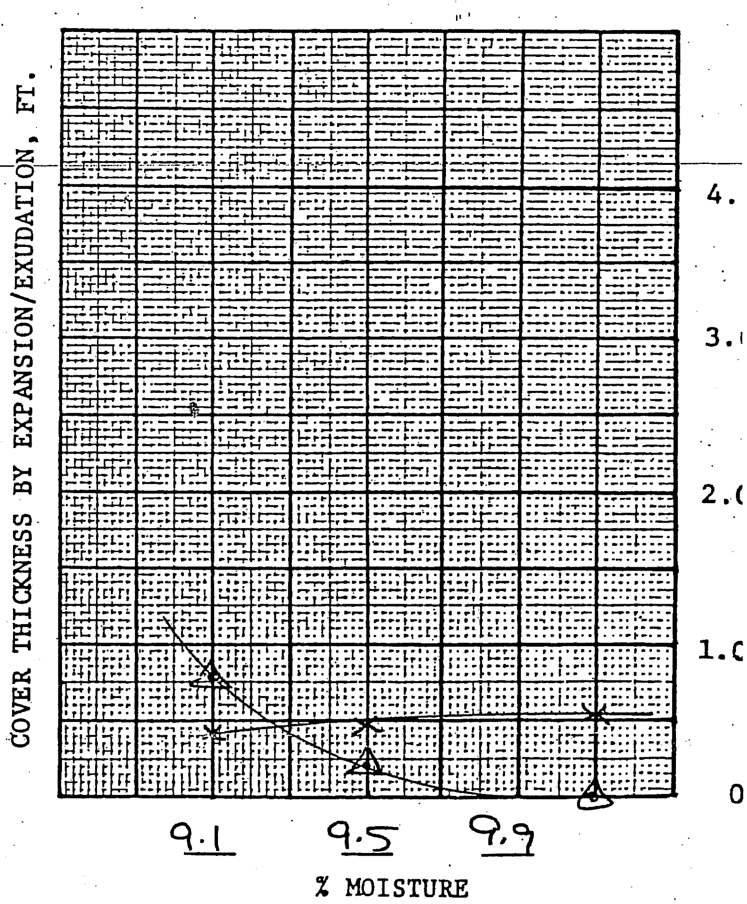
TRAFFIC INDEX Assume 4.0

R-VALUE BY EXUDATION 51

R-VALUE BY EXPANSION







R-VALUE vs. EXUD. PRES.

T by EXUDATION

EXUD. T vs. EXPAN. T

T by EXPANSION

REMARKS

GF=1.25

LaBelle • Marvin
PROFESSIONAL PAVEMENT ENGINEERING

FIGURE 7.2

M.J. SCHIFF & ASSOCIATES, INC.

Consulting Corrosion Engineers - Since 1959
431 W. Baseline Road
Claremont, CA 91711

Phone: (909) 626-0967 / Fax: (909) 626-3316
E-mail: mjsa@mjschiff.com
http://www.mjschiff.com

October 21, 2004

MACTEC
200 Citadel Drive
Los Angeles, CA 90040

Attention: Mr. J. Adolfo Acosta, Ph.D., G. E.

Re: Soil Corrosivity Study
Long Beach Memorial Hospital
Southwest Corner of Long Beach Boulevard and Spring Street
Long Beach, California
Your # 4953-04-2891-02, MJS&A #04-1413HQ

INTRODUCTION

Laboratory tests have been completed on two soil samples you provided for the referenced project. The purpose of these tests was to determine if the soils might have deleterious effects on underground utility piping, hydraulic elevator cylinders, and concrete structures. We assume that the samples provided are representative of the most corrosive soils at the site.

The proposed project is construction of a 3-story building. The water table is 53 feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. Our recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, we will be happy to work with them as a separate phase of this project.

TEST PROCEDURES

The electrical resistivity of each sample was measured in a soil box per ASTM G57 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soils and for ammonium and nitrate. Test results are shown in Table 1.

SOIL CORROSIVITY

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:

Soil Resistivity in ohm-centimeters			Corrosivity Category
over		10,000	mildly corrosive
2,000	to	10,000	moderately corrosive
1,000	to	2,000	corrosive
below		1,000	severely corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivities were in the moderately corrosive category with as-received moisture. When saturated, the resistivities were in the corrosive category.

Soil pH values were 7.6 and 7.8, which are mildly alkaline.

The soluble salt content of the samples ranged from low to moderate.

The nitrate concentration was high enough to be deleterious to copper.

Tests were not made for sulfide and negative oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as corrosive to ferrous metals and aggressive to copper.

CORROSION CONTROL RECOMMENDATIONS

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

Steel Pipe

Abrasive blast underground steel piping and apply a dielectric coating such as polyurethane, extruded polyethylene, a tape coating system, hot applied coal tar enamel, or fusion bonded epoxy intended for underground use.

Bond underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

Electrically insulate each buried steel pipeline from dissimilar metals and metals with dissimilar coatings (cement-mortar vs. dielectric), and above ground steel pipe to prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection.

Apply cathodic protection to steel piping as per NACE International Standard RP-0169-02.

As an alternative to dielectric coating and cathodic protection, apply a ¾ inch cement mortar coating or encase in concrete 3 inches thick, using any type of cement.

Hydraulic Elevator

Coat hydraulic elevator cylinders as described above. Electrically insulate each cylinder from building metals by installing dielectric material between the piston platen and car, insulating the bolts, and installing an insulated joint in the oil line. Apply cathodic protection to hydraulic cylinders as per NACE International Standard RP-0169-02. As an alternative to electrical insulation and cathodic protection, place each cylinder in a plastic casing with a plastic watertight seal at the bottom.

The elevator oil line should be placed above ground if possible but, if underground, should be protected as described above for steel utilities, or should be placed in a PVC casing pipe to prevent contact with soil and soil moisture.

Iron Pipe

Pressurized Pipe:

Encase pressurized cast and ductile iron piping per AWWA Standard C105 or coat with epoxy or polyurethane intended for underground use. Note: the thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating. Electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE International Standard RP-0286-02. Bond all nonconductive type joints for electrical continuity. Apply cathodic protection to cast and ductile iron piping as per NACE International Standard RP-0169-02.

Non-Pressurized Pipe (Consider one of the following alternatives for protection):

1. Polyethylene encase cast- and ductile-iron piping per AWWA Standard C105. Electrically insulate underground pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE International Standard RP-0286-02. Protect all non-cast iron and non-ductile iron fittings and valves with wax tape per AWWA Standard C217-99 after assembly.
2. Concrete encase all buried portions of metallic piping so that there is a minimum of 3-inches of concrete cover provided over and around surfaces of pipe, fittings, and valves.
3. Apply cathodic protection to cast and ductile iron piping as per NACE International Standard RP-0169-02.

Copper Tube

Buried copper tubing shall be protected by:

1. Encasing the copper in two layers of 10-mil thick polyethylene sleeves taking care not to damage the polyethylene. Protect wrapped copper tubing by applying cathodic protection per NACE International Standard RP-0169-02. Any damaged polyethylene shall be repaired by wrapping it in 20-mil thick pipe wrapping tape. The amount of cathodic protection current needed can be minimized by coating the tubing.
2. Preventing soil contact. Soil contact may be prevented by placing the tubing above ground.
3. Install a factory coated copper pipe with a minimum of 100-mil thickness such as "Aqua Shield" or similar products. Polyethylene coating protects against elements that corrode copper and prevents contamination between copper and sleeving. However, it must be continuous with no cuts or defects if installed underground.

Plastic and Vitrified Clay Pipe

No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint. Protect all fittings and valves with wax tape per AWWA Standard C217-99 or epoxy.

All Pipe

On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA Standard C217-99 after assembly.

Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete

Any type of cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.1 percent, per 1997 Uniform Building Code (UBC) Table 19-A-4 and American Concrete Institute (ACI-318) Table 4.3.1.

Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils.

CLOSURE

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
M.J. SCHIFF & ASSOCIATES, INC.

Adrineh Avedisian

Reviewed by,

John W. French, P. E.

REGISTERED PROFESSIONAL ENGINEER
JOHN W. FRENCH
No. CR001023
2/3/05
CORROSION
STATE OF CALIFORNIA

Enc: Table 1

M. J. Schiff & Associates, Inc.
Consulting Corrosion Engineers - Since 1959
431 W. Baseline Road
Claremont, CA 91711
Phone: (909) 626-0967 Fax: (909) 626-3316
E-mail lab@mjschiff.com
website: mjschiff.com

Table 1 - Laboratory Tests on Soil Samples

Long Beach Memorial Hospital, Long Beach, CA
Your #4953-04-2891.02, MJS&A #04-1413HQ
8-Oct-04

Sample ID			4 @ 8' (ML - CL)	3 @ 3' (ML)
Resistivity		**Units**		
as-received		ohm-cm	3,400	5,300
saturated		ohm-cm	1,700	1,500
pH			7.6	7.8
Electrical Conductivity		mS/cm	0.14	0.20
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	24	24
magnesium	Mg^{2+}	mg/kg	58	32
sodium	Na^{1+}	mg/kg	ND	67
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	98	101
chloride	Cl^{1-}	mg/kg	ND	35
sulfate	SO_4^{2-}	mg/kg	128	197
Other Tests				
ammonium	NH_4^{1+}	mg/kg	1.7	1.5
nitrate	NO_3^{1-}	mg/kg	ND	31.7
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

Appendix E

Geology and Soils

Environmental Consultants

3711 Long Beach Boulevard
9th Floor
Long Beach, California 90807

562-426-9544
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SCS ENGINEERS

October 7, 2004
Project No. 01203219.01

Nuna Tersibashian
Sapphos Environmental, Inc.
133 Martin Alley
Pasadena, CA 91105

SUBJECT: GEOLOGY AND SOILS SECTION, LONG BEACH MEMORIAL MEDICAL CENTER EXPANSION, ENVIRONMENTAL IMPACT REPORT

Dear Ms. Tersibashian;

Attached is the final version of the Technical Appendix on Geology and Soils for the Long Beach Memorial Medical Center Expansion Project, Environmental Impact Report. An additional copy of the final report was e-mailed to your attention on October 7, 2004.

Please let me know if you have any additional comments or questions regarding this information at 562-426-9544.

Sincerely,

Heather Tomley

Heather Tomley
Project Scientist
SCS ENGINEERS

Enclosure

GEOLOGY AND SOILS

To Accompany:

LONG BEACH MEMORIAL MEDICAL CENTER EXPANSION ENVIRONMENTAL IMPACT REPORT

Prepared by:

SCS Engineers
3900 Kilroy Airport Way
Suite 100
Long Beach, California 90806
(562) 426-9544

December 2004
File No. 01203219.01

Table of Contents

1.0 GEOLOGY AND SOILS 1

1.1 Regulatory Framework 1

1.2 Existing Conditions...4

1.3 Significant Thresholds 7

1.4 Impact Analysis 7

1.5 Mitigation Measures 8

1.6 Level of Significance After Mitigation...9

1.7 References..9

Figures

1 Geological Map, LBMMC Expansion Project Area, Long Beach, CA.

2 Active Faults, Surface Expression, Los Angeles Basin.

3 Liquefaction Hazard Zone from California State Seismic Hazard Map, LBMMC Expansion Project Area, Long Beach, CA.

4 Approximate Location of Unclassified Fill, LBMMC Expansion Project Area, Long Beach, CA.

Tables

1 Major Named Faults Considered to be Active in Southern California.

2 Major Named Faults Considered to be Potentially Active in Southern California.

3 Historical Earthquakes of Magnitude 6.0 and Above within 100 km of the Project.

1.0 GEOLOGY AND SOILS

As a result of the Initial Study, the City of Long Beach determined that the proposed project had the potential to result in impacts related to geology and soils. Detailed analysis of this issue is, therefore, included in this Environmental Impact Report (EIR). This analysis was undertaken to identify opportunities to avoid, reduce, or otherwise mitigate potentially significant impacts related to geology and soils.

This analysis has been undertaken to determine if the LBMMC Expansion project (proposed project) may have a significant impact on geology and soils in accordance with Section 15063 of the CEQA Guidelines. The conclusions rely upon expert opinion supported by facts, published maps and reports (such as California Geological Survey, formerly Department of Conservation Division of Mines and Geology, 1988a, 1994, 1996, 1997a, 1999; U.S. Geological Survey, 1964), technical studies, and planning documents such as the County of Los Angeles General Plan Safety Element (County of Los Angeles Department of Regional Planning, 1990), and the City of Long Beach Municipal Code (Building Codes, Municipal Code Chapter 18.24) and General Plan Seismic Element (1988). Technical analysis for this section of the EIR was completed by SCS Engineers (SCS).

1.1 Regulatory Framework

This regulatory framework identifies the federal, state, and local statutes and policies that relate to geology and soils and must be considered during the decision-making process for projects that involve grading (excavation or fill), modification of existing structures, or construction of new structures.

State

California Geological Survey (CGS)

The CGS identifies several earth resource issues that should be taken into consideration when evaluating whether the proposed project would likely be subject to geologic hazards, particularly hazards related to earthquake damage. These considerations include both the potential for existing geologic and soil conditions to pose a risk to the project and the potential for the proposed project to result in an impact to the existing geologic and soil conditions by creating or exacerbating a geologic hazard.

The CGS conducts studies related to geologic hazards (e.g., faulting, liquefaction, seismically induced landslides, and ground shaking) as they affect people and structures. These relate to the Alquist-Priolo Earthquake Fault Zone (APEFZ) Act and Seismic Hazards Mapping Act, described below. The CGS also issues guidelines for the evaluation of geologic and seismic factors that may impact a project or that a project may affect. The CGS publications that are most applicable are as follows:

- Special Publication 99, Planning Scenario for a Major Earthquake on the Newport-Inglewood Fault Zone (CGS 1988a).

- Open File Report 88-14, Recently Active Strands of the Newport-Inglewood Fault Zone, Los Angeles and Orange Counties, California, (CGS 1988b).

- Special Publication 42, Fault Rupture Hazard Zones in California (CGS, revised 1997).
- Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California (CGS, 1997).

Alquist-Priolo Earthquake Fault Zone Act of 1972

The CGS has delineated Earthquake Fault Zones along known active or potentially active faults in California pursuant to the APEFZ Act of 1972 (Public Resources Code Section 2621 et seq.). This designation indicates that an active fault is present in the zone and may pose a risk of surface rupture. The State of California (State) delegates the authority to local government to regulate development within APEFZ. Construction of habitable structures is not permitted over areas of potential rupture. A geologic study would likely be required prior to construction of such structures within Earthquake Fault Zones to demonstrate that they are not located over an area of potential rupture.

The closest active fault to the project site is the Cherry Hill segment of the Newport-Inglewood fault. Recent information indicates that the fault is located approximately 1000 feet (300 meters) northeast of the project site (personal communication D. Clarke, City of Long Beach Department of Oil Properties, 2004). Proposed project buildings are not identified as being within an Earthquake Fault Zone on an APEFZ Map. Therefore, the potential for surface rupture due to fault plane displacement propagating to the surface under structures during the design life of the project is considered low.

Seismic Hazards Mapping Act of 1990

The CGS has also identified Seismic Hazard Zones that are delineated in accordance with the Seismic Hazards Mapping Program of the Seismic Hazards Mapping Act of 1990 (Public Resources Code Section 2690 et seq.). The Act provides for "a statewide seismic hazard mapping and technical advisory program to assist cities and counties in fulfilling their responsibilities for protecting the public health and safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure, and other seismic hazards caused by earthquakes." Portions of the proposed project site are identified on the Long Beach Quadrangle Official Seismic Hazard Zones Map within a zone of liquefaction potential (CGS, 1999).

California Building Code

The majority of coastal California, including the proposed project site, lies within Seismic Zone 4, the highest level hazard zone designated by the current Uniform Building Code (UBC). The California Building Standards Code, or California Building Code (CBC), augments and supercedes the UBC with stricter requirements to reduce the risks associated with building in Seismic Zone 4 to the maximum extent practicable. The CBC (Code of Regulations, Title 24) sets standards for the investigation and mitigation of the site conditions related to fault movement, liquefaction, landslides, differential compaction/seismic settlement, ground rupture, ground shaking, tsunami, seiche, and seismically induced flooding. Mitigation of geological (including earthquake) and soil (geotechnical) issues must be undertaken in compliance with the CBC.

Local

County of Los Angeles General Plan

The County of Los Angeles Board of Supervisors adopted the first Safety and Seismic Safety elements in 1975 as components of the County of Los Angeles General Plan (County of Los Angeles, 1980, 1990). The provisions of those elements were updated, revised, and combined in one document and included in the Streamlined County of Los Angeles General Plan (County of Los Angeles, 1993). The current County Safety element addresses seismic hazards related to surface rupture and ground shaking, as well as geologic hazards associated with unstable ground.

Specifically, the General Plan includes a seismic hazard goal to minimize injury and loss of life, property damage, and the social, cultural, and economic impacts caused by earthquake hazards. The policies supporting this goal relevant to the proposed project, include continue enforcement of stringent site investigations (such as seismic, geologic, and soils investigations) and implementation of adequate hazard mitigation measures for development projects in areas of high earthquake hazard. The "Seismic Zones Map" of the County of Los Angeles General Plan must be taken into consideration in the project planning process.

The geologic hazards goal to protect public safety and minimize the social and economic impacts from geologic hazards is supported by policies relating to issues such as approval of projects in areas that are susceptible to landsliding, debris flow, and rockfalls and in areas where collapsible soils are problems. Approval in these cases is contingent on the ability to satisfactorily mitigate these problems.

County of Los Angeles Building Codes

The County has adopted and amended the California Building Code, described above, to reflect local geologic and seismic conditions. The County of Los Angeles Building Code (Los Angeles County Codes, Title 26) would be the standard for evaluation of the adequacy of geotechnical and engineering geology studies needed for design and construction in the County. The proposed project would be subject to the provisions of both the CBC and the County of Los Angeles Building Code. In addition, the Building and Safety Division of the County of Los Angeles Department of Public Works has jurisdiction over projects where grading is required to ensure the safety of workers and to ensure the safety of the public once the project is constructed. Grading and proposed structures must comply with the County Building Code and the County Hydrology Manual.

City of Long Beach

Building and construction within the City of Long Beach are subject to the regulations of the City of Long Beach Municipal Code. Municipal Code Chapter 18.24, Building Codes, adopts and incorporates by reference the California Building Code (Volumes I and II, 2001 Edition). This Municipal Code chapter includes amendments and modifications to the California Building Code that are specific to Long Beach. The California Building Code in turn incorporates provisions of the Uniform Building Code, which contains seismic design criteria and grading standards.

The City of Long Beach General Plan adopted the Seismic Safety Element of the General Plan on October 1988. The purpose of this element is to provide a comprehensive analysis of seismic factors in order to reduce the loss of life, injuries, damage to property, and social and economic impacts resulting from future earthquakes. The Seismic Safety Element is a seismic safety planning tool and contains goals and recommendations that provide guidance for development in seismically active areas. To achieve maximum feasible safety from seismic risk, the Seismic Safety element focuses upon current developmental policies as well as the allocation of future land uses.

1.2 EXISTING CONDITIONS

Regional Geology

Geologically, the project area is located in the southwestern portion of the Los Angeles basin. The basin formed when basement (older) rocks were structurally downwarped allowing a thick sequence of Upper Cretaceous through Recent age (approximately 100 million years ago to present) sedimentary units to form. The sedimentary basin fill in the project area is estimated to be 12,000 feet (3660 meters) thick (Yerkes, et al, 1965). The basin fill in this area consists predominantly of marine origin sandstone, siltstone, and shale of Middle Miocene to Pliocene age (approximately 16 to 1.8 million years ago) overlain by predominantly marine sand and silt of Pleistocene to Recent age (approximately 1.8 million years ago to present).

The rocks of the basin are cut by numerous faults, many of which are strike-slip faults of generally northwest-southeast orientation. A number of faults in the basin are considered to be active or potentially active (Tables 1 and 2). Research has also indicated several blind thrust faults (low angle faults which do not break the surface) are active or potentially active and could cause significant ground shaking (Shaw and Suppe, 1996). Of faults considered active, the Newport-Inglewood fault zone is located closest to the project site, within approximately 1,000 feet (300 meters) northeast. The

Newport-Inglewood zone extends from the Baldwin Hills to Newport Bay and is considered active. Some recent research also indicates that the Compton-Los Alamitos Blind Thrust, which may be located in the deep subsurface under the project site, may or may not be active or potentially active (Shaw and Suppe, 1996; Mueller, 1997).

Site Specific Geology and Soils

Surface elevation in the project area is between approximately 35 and 50 feet (10 to 15 meters) above mean sea level. The site generally slopes to the southwest but there are no steep slopes. The investigation area is located on the western flank of the Signal Hill uplift, approximately 1 mile (1.6 kilometers) east of the Los Angeles River and approximately 3 miles (4.8 kilometers) north of the Long Beach shoreline. Surficial geologic materials in the area consist of Pleistocene and Recent non-marine and marine units, predominantly sand, silty sand, sandy silt, silt, and clay (Figure 1). Undisturbed soil at the site is not considered significantly erodable. In addition to native materials and engineered fill placed in connection with construction activities, an unknown volume of unclassified fill, including gravel, debris, and waste oil field material, was used to bring a former on-site ravine up to grade prior to use of the site for hospital facilities.

Native and fill soils were encountered in borings drilled during subsurface site investigations. There are no unique geological features at the project site.

Portions of the project area are within the Long Beach oil field and several abandoned petroleum production wells are located at the site (see detailed description of these in the Hazardous Materials section of this EIR). The portion of the ground surface within the oil field that is also within the project area no longer contains active oil production facilities. The project is not located in a Mineral Resource Zone as identified by the CGS.

The uppermost regional aquifer in this area is anticipated to be the Gage (California Department of Water Resources, 1961), located at a depth of approximately 200 to 250 feet (60 to 75 meters) below ground surface (bgs). Uppermost groundwater beneath most of the area occurs at a depth estimated at 50 feet (15 meters) bgs within sands of the Lakewood Formation, however a thin perched zone of groundwater was encountered as shallow as 15 feet (5 meters) bgs in the northern portion of the expansion area.

As indicated above, a portion of the Newport-Inglewood fault zone, known as the Cherry Hill segment, is located within approximately 1,000 feet (300 meters) of portions of the project area. The Newport-Inglewood fault is capable of a 7.1 magnitude earthquake (Cao, et al, 2003). Maximum horizontal ground acceleration was estimated on a design and upper bound earthquake basis in a recent study (MACTEC, 2004), with a 10 percent chance of exceedance during 50- and 100-year time periods, respectively. The design and upper bound basis peak ground accelerations were estimated at 0.52 and 0.65 g.

Environmental Setting in Relation to Proposed Project

Seismicity --

The project is located in an area that is susceptible to strong ground shaking from severe earthquakes. Earthquakes on faults, such as the nearby Newport-Inglewood (capable of 7.1 magnitude), can generate seismic shaking. There are also a number of other active and potentially active faults within 60 miles (100 kilometers) of the site, any of which could cause significant ground shaking at the site (Tables 1 and 2, Figure 2). Some of the faults present a risk of very strong ground shaking that must be considered for facilities where public safety and post-earthquake function are necessary. Implementation of the proposed project could expose people and structures to strong seismic ground shaking, which represent a potentially significant adverse impact unless mitigation is incorporated.

Potential seismic forces resulting from an earthquake as they might affect buildings and other structures are often quantified as peak ground acceleration. MACTEC (2004) has determined site-specific peak ground acceleration using the Design Basis Earthquake having a 10 percent probability of exceedance during a 50-year time period and the Upper Bound Earthquake having a 10 percent probability of exceedance during a 100-year time period of 0.52 g and 0.65 g, respectively. It is recommended that conservative factors of safety be applied to the design of critical structures.

Groundwater --

Groundwater has been encountered at depths of 40 to 50 feet (12 to 15 meters) below ground surface in the project area. Approximately 10 to 15% of the project site overlies an area potentially susceptible to liquefaction, as indicated on the California State Seismic Hazard Maps. A portion of the site, extending from near the intersection of Columbia Street and Atlantic Avenue in the northeast to the intersection of Patterson Avenue and Long Beach Boulevard on the west, is susceptible to liquefaction (Figure 3). This area is the former location of a ravine crossing the area that was backfilled with unclassified fill soil prior to construction of the present hospital buildings (Figure 4). Some of this unclassified fill has subsequently been removed and replaced by engineered fill. Perched groundwater has been encountered in this fill material (SCS, May 2004). The perched water may be seasonal. Much of this unsuitable fill material has been removed and replaced with compacted engineered fill, however some remains. All of the remaining unclassified fill that underlies buildings that are to be constructed as part of the project will also be removed and replaced. The most common effects of liquefaction are ground settlement and cracking, sinking and/or tilting of heavy surface structures, buoyancy of some buried structures (e.g., pipelines, tanks), and shallow lateral spread landslides near drainages with exposed "free faces" (e.g., flood control channels, stream banks). Based on soil parameters measured at the site, MACTEC (2004) has calculated the liquefaction induced settlement to be less than 0.25 inches (0.64 centimeters). Where liquefaction does not occur, soils may be subject to seismic settlement from densification during severe shaking.

Soils Issues --

Expansive soils have relatively high clay mineral content and are usually found in areas where underlying formations contain an abundance of clay minerals or where coarse-grained materials are weathered and break down into clay-rich materials. Although there is some clay in the natural soils in the project area, the soil is primarily silt and silty sand. The Leroy Crandall and Associates report of April 10, 1969 indicates that the clay soils are somewhat expansive. Following standard engineering practice, all expansive soil that could potentially negatively affect buildings or other project components would be removed and replaced with properly engineered fill soil prior to building construction.

As described in the project report Hazards and Hazardous Materials (SCS, October 2004), evaluation of California Department of Conservation, Division of Oil, Gas, and Geothermal Resources (DOGGR) records for the project area revealed 9 former oil well locations on the project site. Activities associated with oil well drilling and oil production, including drilling mud pits, sumps, and pipelines, may be encountered in the vicinity of the former wells. Some of these facilities may be associated with soil contaminated with hydrocarbons, metals, or other potentially hazardous substances. As discussed in the Hazards and Hazardous Materials report, soil with field indications of potential contamination encountered during project earthwork, will be tested and removed if found to be contaminated or otherwise unsuitable. This approach will apply also to soils, described above as unclassified fill, located in a former ravine that was historically filled using petroleum containing soil and miscellaneous oil field and other debris (see Figure 4 for approximate location of remaining unclassified fill).

1.3 Significance Thresholds

The following are the potentially relevant standards of significance:

- The project would conflict with legal requirements regarding geologic hazards or soil conservation.
- The project would expose people or structures to significant injury or damage due to geological hazards. For instance if the project is located in a known fault rupture zone, the site includes material subject to seismically induced liquefaction or landsliding, or the soils are sufficiently expansive or likely to subside so that significant building damage might result.
- The project would result in significant soil erosion, loss of mineral resources, or loss of a unique geological feature.

1.4 Impact Analysis

Seismicity

The surface expression of known active or potentially active faults do not pass directly through the project site, however a number of known regional active faults are located at distances where they could produce substantial ground shaking at the project site. Similar to development throughout most of southern California, implementation of the proposed project will result in exposure of persons at the project site to substantial ground shaking and thus a degree of seismic hazard risk. The proposed project will be constructed in accordance with the California Building Code, Long Beach Municipal Code, and Uniform Building Code. In addition, the maximum probable seismic ground acceleration will be taken into consideration when designing all structures in order to minimize potential hazards. Furthermore, geotechnical studies prepared for each phase of building will be undertaken in accordance with the CGS Guidelines for Evaluating and Mitigating Seismic Hazards in California (1997b). The project will be consistent with the goals and recommendations of the Seismic Safety Element of the Long Beach General Plan. For these reasons, impacts associated with seismic hazards will be reduced to the extent possible and will be less than significant.

Another potential impact associated with seismic activity that could occur at the site is liquefaction. Soils in most areas of the site are not susceptible to liquefaction, however, as discussed previously, some areas of the project site are located within the CGS liquefaction hazard zone. Potential impacts due to liquefaction could include foundation bearing failure or large foundation settlements, imposition of additional loads on foundations, localized lateral displacement (spreading) or compression, floatation of light structures, and damage to infrastructure such as streets and utilities. The liquefaction potential will be evaluated as part of the detailed geotechnical study for each new building phase and for any new infrastructure, as required by the California Building Code and Uniform Building Code. Unsuitable fill soils located under proposed structures will be removed and replaced with properly engineered fill. Subsurface drainage will be provided where necessary to prevent near surface soil saturation. Geotechnical studies and design will be undertaken in accordance with the CGS Guidelines for Evaluating and Mitigating Seismic Hazards in California (1997b). For these reasons, impacts associated with potential liquefaction will be less than significant.

Grading-Related

Since the project will, overall, require substantial grading and filling, erosion or stockpiled soil or of exposed soil surfaces could occur. Specifically, excavation, grading, stockpiling, and other earth moving activities could exposed site soils to wind- or water-generated erosion. Best management practices will be employed in preparation of and during periods of precipitation when earth moving activities are being conducted or when soil has been exposed by these activities in order to eliminate or reduce erosion to the extent possible. Implementation of Best Management Practices will ensure that the project will not result in substantial soil erosion and, therefore, erosion related impacts will be less than significant.

Although there are some clay soils in the project area and some of these have been indicated to be somewhat expansive in a few past geotechnical reports, soil at the site is predominantly silty and sandy and not expansive. Extensive additional geotechnical testing will be conducted in area where foundations will be placed and any unsuitable soils that could potentially swell and affect buildings or other proposed project components will be removed and replaced with engineered fill. In addition, there is no evidence or reason to believe that soils, with the possible exception of unclassified fill, would be subject to significant subsidence. For these reasons, impacts associated with potentially expansive soils will be less than significant.

As indicated above, if indications, of potentially contaminated soil, such as discoloration or odors, are encountered during grading, these soils will be tested and removed if found to be unsuitable to remain in place. Additional information on this topic is included in the section of the EIR dealing with Hazards and Hazardous Materials.

Cumulative Impacts

Development of the LBMMC expansion will not result in cumulatively significant impacts associated with potential seismic hazards or with grading.

1.5 Mitigation Measures

The following mitigation measures are recommended to ensure that potential impacts associated with geology and soils will be less than significant:

- **Measure Geology-1**. Geotechnical studies will be conducted as necessary to assure that all critical soil parameters are defined, areas and depths where soil saturation could occur are delineated, and maximum probable ground acceleration and other seismic related effects can be predicted.

- **Measure Geology-2**. Design will be in accordance with the Seismic Safety Element of the Long Beach General Plan, the California Building Code, and the Uniform Building Code. Design will take into account all data resulting from geotechnical studies, including items such as anticipated maximum seismic ground acceleration, soil bearing strength, and optimal soil compaction parameters. The foundation design will also include specifications for removing all of the remaining unclassified fill that underlies buildings that are to be constructed as part of the project.

- **Measure Geology-3**. Construction oversight will be conducted in order to assure that project elements are built in accordance with the design. If unexpected geologic or soils elements are encountered during excavation construction oversight will assure that these are defined and incorporated into design modifications, as necessary.

- **Measure Geology-4**. Best Management Practices will be implemented during times when soils are exposed to precipitation in order to minimize erosion. Best Management Practices will be in accordance with California State Water Resources Control Board and U.S. EPA guidance and may include such items as use of sediment traps and filters during construction.

1.6 Level of Significance After Mitigation

With implementation of the recommended mitigation measures, potentially significant impacts due to geology and soil related issues will be less than significant. Conduct of geotechnical studies of adequate scope and number, a number of which have already been completed, will assure that subsurface geology and soils engineering issues are well defined so that design can rely on the data generated. Design in accordance with Long Beach General Plan and Building Codes, the California Building Code, and the Uniform Building Code will assure that the resulting project elements will provide the maximum protection against seismic hazards. Construction oversight will assure that design elements are met. Implementation of Best Management Practices will insure that soil erosion is less than significant.

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FIGURES

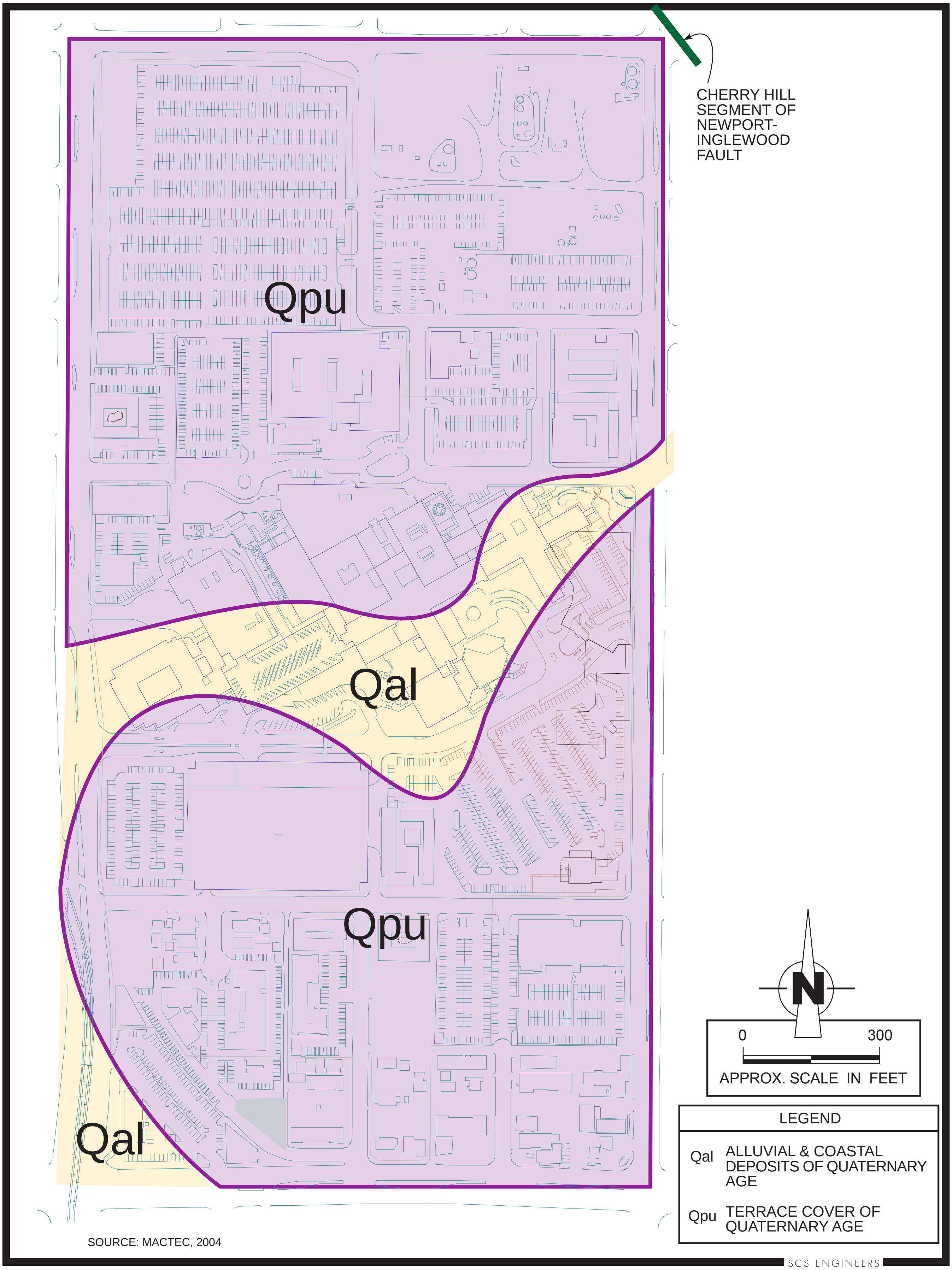

Figure 1. Geological Map, LBMMC Expansion Project Area, Long Beach, CA.

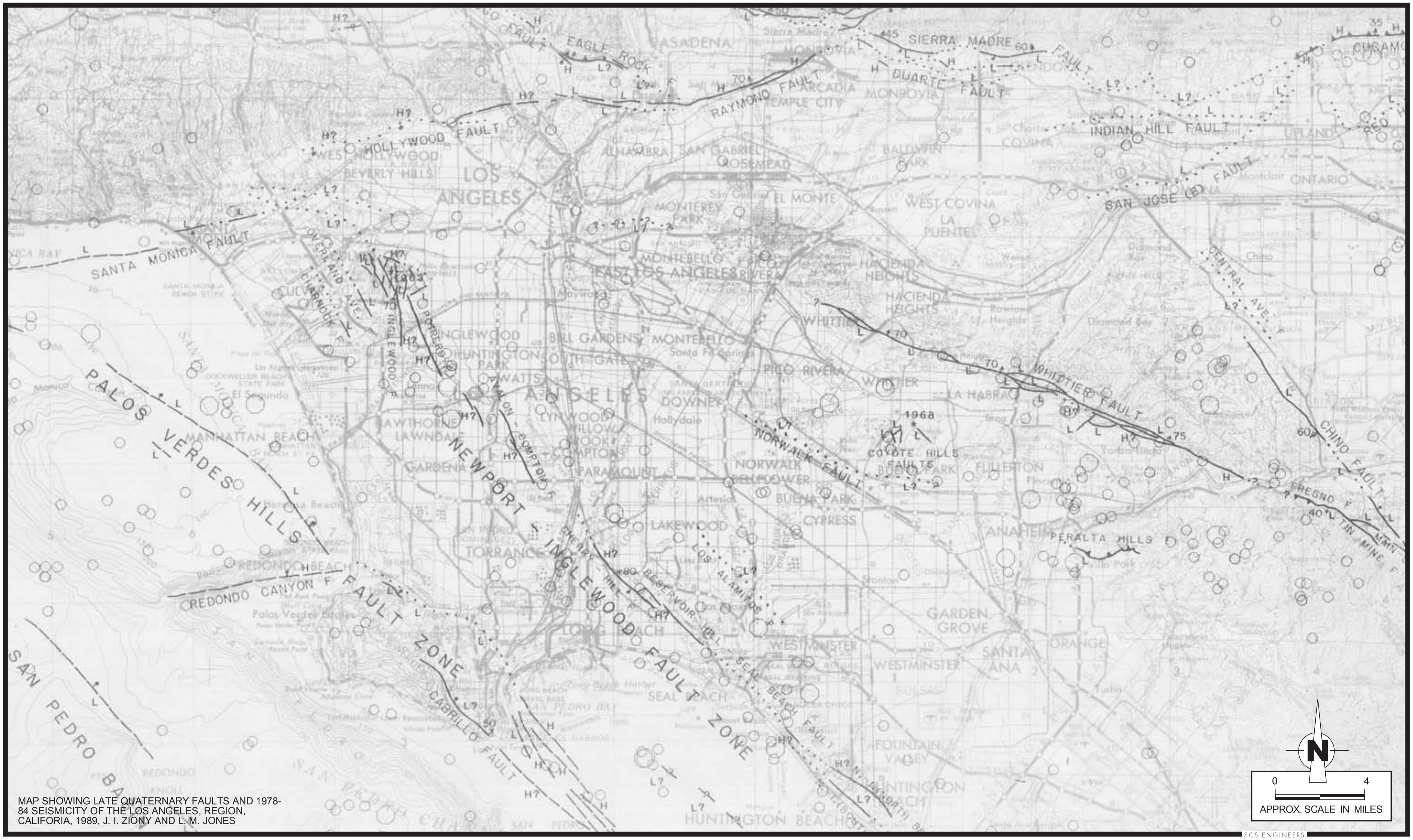

Figure 2. Active Faults, Surface Expression, Los Angeles Basin.

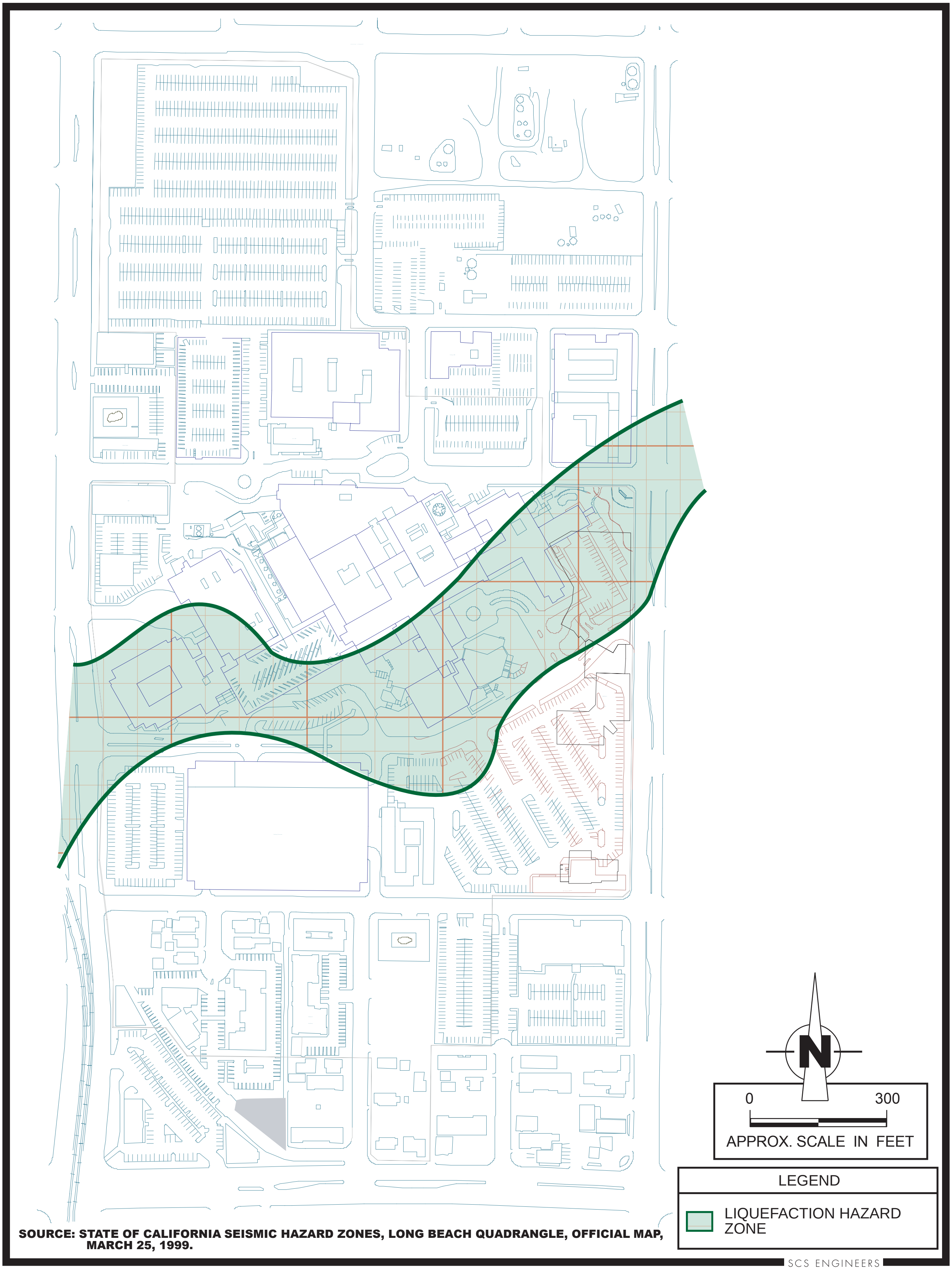

Figure 3. Mapped Liquefaction Hazard Zone from California State Seismic Hazard Map, LBMMC Expansion Project Area, Long Beach, CA.

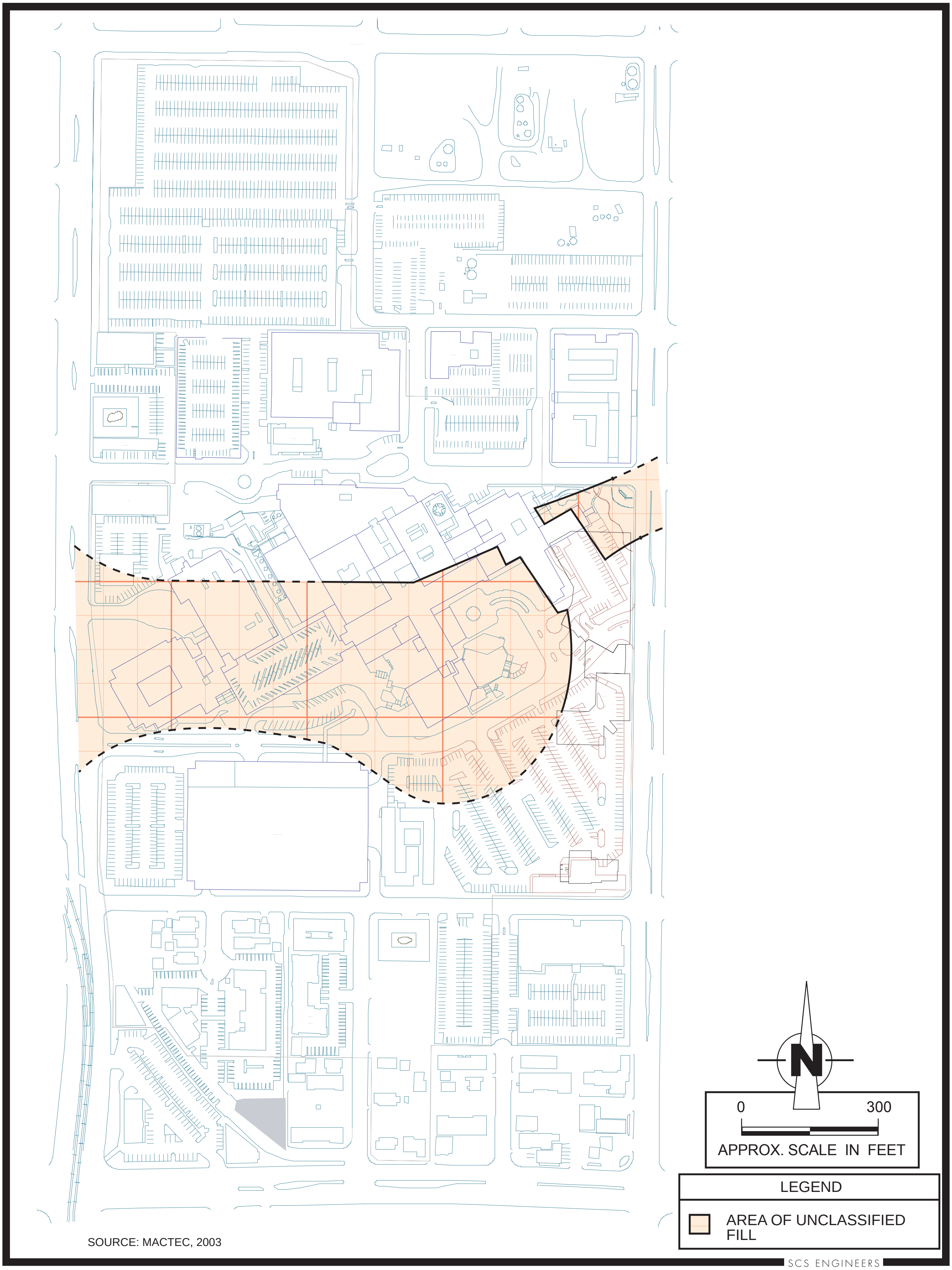

Figure 4. Area of Unclassified Fill, LBMMC Expansion Project Area, Long Beach, CA.

TABLES

Table 1
Major Named Faults Considered to be Active
in Southern California

Fault (increasing distance)	Maximum Magnitude	Fault Type	Slip Rate (mm/yr.)	Distance from Site (kilometers)	Direction from Site
Newport-Inglewood Zone	7.1	SS	1.0	0.3	NE
Palos Verdes	7.3	SS	3.0	10 ½	SW
Puente Hills Blind Thrust	7.1	BT	0.7	11 ½	NE
Upper Elysian Park	6.4	BT	1.3	25	N
Whittier	6.8	SS	2.5	25	NE
San Joaquin Hills	6.6	BT	0.5	27	SW
Santa Monica	6.6	RO	1.0	33	NW
Raymond	6.5	RO	1.5	34	N
Hollywood	6.4	RO	1.0	36	N
Verdugo	6.9	RO	0.5	38	N
Malibu Coast	6.7	RO	0.3	41	NW
Sierra Madre	7.2	RO	2.0	42	NE
Northridge Thrust	7.0	BT	1.5	45	NW
Chino - Central Avenue	6.7	NO	1.0	49	NE
Elsinore (Glen Ivy Segment)	6.8	SS	5.0	49	E
San Gabriel	7.2	SS	1.0	51	NNE
Anacapa-Dume	7.5	RO	3.0	52	NW
San Fernando	6.7	RO	2.0	52	N
Cucamonga	6.9	RO	5.0	57	NE
Simi-Santa Rosa	7.0	RO	1.0	72	NW
San Andreas (San Bernardino Segment)	7.5	SS	24.0	78	NE
Oak Ridge	7.0	RO	4.0	79	NW
San Jacinto (San Bernardino Segment)	6.7	SS	12.0	79	NE
San Cayetano	7.0	RO	6.0	100	NW

SS - Strike Slip
NO - Normal Oblique
RO - Reverse Oblique
BT - Blind Thrust

Source: MACTEC, 2003

Table 2
Major Named Faults Considered to be Potentially Active
in Southern California

Fault (increasing distance)	Maximum Magnitude	Fault Type	Slip Rate (mm/yr.)	Distance from Site (kilometers)	Direction from Site
Los Alamitos	6.2	SS	0.1	6.7	NE
Norwalk	6.7	RO	0.1	15	NE
Charnock	6.5	SS	0.1	25	NW
Coyote Pass	6.7	RO	0.1	25	N
Overland	6.0	SS	0.1	26	NW
MacArthur Park	5.7	RO	0.1	27	N
Clamshell-Sawpit	6.5	RO	0.5	43	NE
Duarte	6.7	RO	0.1	43	NNE
San Jose	6.4	RO	0.5	44	NE
Indian Hill	6.6	RO	0.1	45	NE
Northridge Hills	6.6	SS	1.2	53	NW
Santa Susana	6.7	RO	5.0	63	NW
Holser	6.5	RO	0.4	81	NW

SS - Strike Slip
NO - Normal Oblique
RO - Reverse Oblique
BT - Blind Thrust

Source: MACTEC, 2003

Table 3
Historical Earthquakes of Magnitude 6.0 and Above
within 100 km of the Project

Date	Location (latitude, longitude)	Local Magnitude *	Moment Magnitude*	Distance from Site (kilometers)
December 8, 1812	33.70, -117.90	6.90 **	7.5 **	33
July 22, 1899	34.30, -117.50	6.50 **	--	85
May 15, 1910	33.70, -117.40	6.00	--	78
July 23, 1923	34.00, -117.25	6.25	--	92
March 11, 1933	33.62, -117.97	6.30	6.4	33
February 9, 1971	34.41, -118.40	6.40	6.6	66
October 1, 1987	34.06, -118.08	6.10	5.9	29
February 28, 1990	34.14, -117.70	6.20	--	59
January 17, 1994	34.21, -118.54	6.80 ***	6.7	51

Source: U.S. Geological Survey, Earthquake Hazards Program, National Earthquake Information Center, Earthquake Database; http://neic.usgs.gov/neis/epic/epic_circ.html

* Moment magnitude is preferred to local, or Richter, magnitude because it provides a more reliable estimate of the size of an event, particularly for very large earthquakes.

** Estimated   *** Surface-wave magnitude